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PROCESS DESIGN MANUAL FOR SUSPENDED SOLIDS REMOVAL

Burns and Roe, Incorporated

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Technology Transfer

By
Burns and Roe, Inc.
700 Kinderkamack Road
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This manual comprises a compilation of information on the practice of suspended solids removal from municipal wastewaters. General engineering considerations are cited with respect to their impact on the design of treatment facilities. Specific processes utilized for suspended solids removal are described, discussed and illustrated through the use of data from installations which have employed these processes. Current technology and advanced methods of treatment are stressed in order to provide usable information for implementation in design of new treatment facilities. Some aspects of the operation and maintenance requirements of the described unit processes are delineated, along with the overall estimated costs of construction and operation. The information and data provided are presented in such a manner that they can be readily incorporated into practice.

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FOREWORD

The formation of the Environmental Protection Agency marks a new era of environmental awareness in America. This Agency's goals are national in scope and encompass broad responsibility in the area of air and water pollution, water supply, solid wastes, pesticides, and radiation. A vital part of EPA's national water pollution control effort is the constant development and dissemination of new technology for wastewater treatment.

It is now clear that only the most effective design and operation of wastewater treatment facilities, using the latest available techniques, will be adequate to meet the future water quality objectives and to ensure continued protection of the Nation's waters. It is essential that this new technology be incorporated into the contemporary design of waste treatment facilities to achieve maximum benefit of our pollution control expenditures.

The purpose of this manual is to provide the engineering community and related industry a new source of information to be used in the planning, design, and operation of present and future municipal wastewater treatment facilities. It is recognized that there are a number of design manuals, manuals of standard practice, and design guidelines currently available in the field that adequately describe and interpret current engineering practices as related to traditional plant design. It is the intent of this manual to supplement this existing body of knowledge by describing new treatment methods, and by discussing the application of new techniques for more effectively removing a broad spectrum of contaminants from wastewater.

Much of the information presented is based on the evaluation and operation of pilot, demonstration and full-scale plants. The design criteria thus generated represent typical values. These values should be used as a guide and should be tempered with sound engineering judgment based on a complete analysis of the specific application.

This manual is one of four now available through the sponsorship of the Environmental Protection Agency to describe recent technological advances and new information in the following subject areas:

Carbon Adsorption
Phosphorus Removal
Upgrading Existing Plants
Suspended Solids Removal

These manuals are the first edition copies and will be updated as warranted by the advancing state-of-the-art to include new data as it becomes available, and to refine design criteria as additional full-scale operational information is generated.

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CHAPTER 1

INTRODUCTION

1.1 General

This manual is designed to provide the consulting engineer and regulatory agencies with up-to-date information on suspended solids removal from wastewaters. Much information required for the design of advanced waste treatment plants is available for many of the processes integral to the proper design of a facility. These data are included in many publications, reports and manufacturers' promotional and design literature. This available information has been considered in the preparation of this manual.

The facilities included in the overall design of a waste treatment plant for the removal of suspended solids should be commensurate with the degree of treatment and process design required. The removal of suspended and/or colloidal solids from wastewater is of major importance in any treatment system. The manual for suspended solids removal does not cover in detail wastewater treatment processes which normally precede or follow the removal of suspended solids except to note the extent to which these processes affect the suspended solids removal processes. In general, the removal of suspended solids from wastewater is accomplished by the following broadly categorized processes:

a. Chemical Processes
b. Gravity Systems
c. Physical Straining
d. In-Depth Filtration

These process categories will be discussed in detail herein.

1.2 Wastewater Solids

Because the term "solids" is so encompassing, some discussion of definitions, assumptions and relationships becomes necessary for proper understanding of the contents of this manual. Wastewater solids are generally classified as total, settleable, suspended, colloidal and dissolved. Total and settleable solids are self-explanatory and are discussed analytically elsewhere (1). Suspended and dissolved solids are differentiated by a standard analytical technique of filtration through a standardized filter (1). The relationship,

$$TS = SS + DS$$

where:

TS = total solids
SS = suspended solids
DS = dissolved solids

is the basis for characterization of wastes as to solid fractions. Any two of the three analyses will determine the relative fractions.

Colloidal solids are an entirely different matter, however. Technically, they can be defined as those particulates with a size range of from 1 to 500 millimicrons (2). In fact, they represent an intermediate fraction between suspended and dissolved solids with some flexibility as to the exact upper and lower size limits. Standard methods of determination are not readily available as in the cases of the other solids fractions. References to the colloidal matter are frequent in the literature in the contexts of discussions concerning coagulation and turbidity of wastewater. Dean, et al. (3), have noted that bacteria, viruses, phages, and other cellular debris fall into the colloidal fraction.

Solids of any of these physically-defined fractions can be further differentiated into organic and inorganic (volatile and ash) or hydrophilic and hydrophobic fractions for example. The need for such classification will vary with individual circumstances. Within the scope of this manual, it is sufficient to note that a differentiation between total and soluble or dissolved fractions of various parameters is of importance. The use of this breakdown is dependent upon an understanding of the processes considered in this text. Certainly, a simple straining or sedimentation design cannot be expected to remove dissolved or soluble pollutants. Consequently, if the desired effluent concentration of such a pollutant is less than the soluble concentration in the feed to a proposed process, one must consider using a process or combination of processes which are capable of removing some of the soluble fraction. This type of judgment becomes possible with proper analytical techniques, such as total and soluble analyses.

The average suspended solids concentration of municipal wastewaters is 200 mg/1. This number is highly variable, depending on a large number of specific circumstances. Contributions from individual homes, multi-family dwellings, or any other point source will often exhibit suspended solids concentrations of a factor of two or more higher than the municipal average (4). Municipal wastewaters have been characterized as to the relative contributions of suspended and dissolved fractions by some investigators. For example, Rawn (5) found that approximately one-third of the total biochemical oxygen demand (BOD) from a subdivision was soluble, while Clark (6) noted less than one-sixth of the BOD from a ski-lodge was dissolved. The residence time in the sewer and other factors which affect the makeup of municipal wastewaters produce a significantly different picture at a central treatment plant. Helfgott, et al (7), found the soluble-colloidal fraction of the chemical oxygen demand (COD) of two New Jersey wastewaters to be 43 percent of the total COD.

Within the scope of this manual, an understanding of the above definitions and relationships will be assumed as basic to all discussions. All wastewaters and subsequent process effluents will exhibit their own individual properties. Any attempt to generalize process capabilities must be treated as approximate and used only for guidance when approaching a specific treatment problem.

1.3 References

1. Standard Methods for the Examination of Water and Wastewater, 13th Edition, American Public Health Association, New York (1971).

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4. Watson, K.S., Farrell, R.P., and Anderson, J.S., "The Contribution from the Individual Home to the Sewer System", Jour. WPCF, 39, 2039 (Dec. 1967).

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CHAPTER 2

POPULATION AND DESIGN PERIOD FORECASTING

2.1 General

This chapter is a rather brief discussion of a highly complex subject concerning the considerations and methods involved in determining proper design periods and flows. One of the most difficult tasks in designing a municipal wastewater treatment facility is the determination of the design capacity of the system. In order to determine design capacity, one must make quantitative projections concerning future values of a number of pertinent factors. Fair and Geyer (1) list the following variables for consideration:

a. Structure and equipment life expectancy and obsolescence.

b. Ease of expansion of facilities.

c. Anticipated contribution population, commercial and industrial growth.

d. Rate of interest on bonded indebtedness.

e. Anticipated inflation rate during retirement period.

f. Facility performance during the early years of underloaded operation.

2.2 Population Growth

Although engineering textbooks and manuals offer a variety of handy mathematical graphical approaches to population forecasting, the most accurate estimate of not only population changes, but also industrial and commercial growth and changes, can usually be found in a city's planning agency. Most urban centers are now required to have master plans and planning agencies in order to obtain federal funding for various projects, most notably urban renewal activities (2). The population projection method commonly used is the "cohort survival model of population dynamics," a most reliable form of forecasting. This method is based on well-established survival and reproductive patterns for all age groups. The major uncertainty is the migration factor, but even this variable can be predictable within limits.

Included in the urban master plan are land use maps indicating the areas intended for housing, types of housing, proposed changes in urban districts, and numerous other data which can be useful to the designer of treatment facilities. The industrial and commercial makeup of the local economy is usually established by an economic survey with predicted changes due to city population, age, density and other factors which exert subtle influences on this makeup.

Where master plans and their associated information are unavailable, the previously-mentioned mathematical models and other pertinent information may be applied to census data to compute some reasonable design population figures. Sources of information for these calculations are numerous in the sanitary engineering literature (1), (3), (4).

2.3 Economic Considerations

As previously noted, interest rates on bonds and anticipated inflation have a bearing on the design period chosen. All governmental bodies have been rated as to their financial status. The corresponding bond rating roughly determines the amount of interest that the city or county will have to pay for the required capital outlay for new facilities. Higher ratings will usually result in lower interest rates. Smith and Eilers (5) indicate that Grade Aaa and Bbb securities had yield rates of 6.0 and 6.5 percent, respectively, in 1970. In essence, during periods of low interest rates, longer design periods may be chosen. This economic analysis may be further influenced by the projected rate of inflation, in that higher inflationary rates mean repayment in dollars of lower value.

2.4 Physical Considerations

When the type, size, and extent of the treatment facilities to be provided have been determined and before design work has physically begun, considerations must be given to the actual positioning of the various units on the available plant site. This aspect must be considered carefully since the physical layout of the plant as a whole may be markedly influenced by the prospects of future plant expansion, alteration or modification on the basis of advancing technology and requirements. The matter of economy alone should not dictate the building of a plant that will only serve to supply immediate needs. Neither is there a valid reason for building a plant so compact that future expansion is virtually ruled out. The basic structures (chemical feed storage and equipment, clarifiers, filters, laboratories, power supply lines and administrative area) will always be required and must, of necessity, be considered as permanent, fixed installations.

Foresight seems to indicate that, with few exceptions, linear construction of a plant is desirable if the facility will be enlarged to any degree in the foreseeable future. Although this type of construction may appear to be excessive in its land requirements, most plants usually find that ease of access to the clarifiers and the filters for the purpose of overhaul and/or maintenance more than compensates for the added distances that must be covered during the course of routine operations.

In some cases, there will be uncertainty as to the precise treatment which is desirable, leading to the suggestion that temporary treatment facilities or structures should be included in the plant. To find any structure ever abandoned, even the temporary ones, is highly unusual. No purely temporary structure should ever be included in the design or construction of a new plant unless short-term usage will make its removal or replacement mandatory. Experience has shown that such structures become very difficult to dispense with and will remain in use even though unsightly, inconvenient and inefficient.

2.5 Wastewater Characteristics

Wastewater flows, chemical and biological characteristics and treatability are necessarily included in treatment plant design procedures. Techniques and other established procedures for accomplishing these tasks are thoroughly covered elsewhere (1),(3),(4). It should suffice for the immediate purposes of this manual to stress the importance of this aspect on the overall plant design, as any failure in this activity can negate all other design considerations.

2.6 References

1. Fair, G.M., and Geyer, J.C., Water Supply and Wastewater Disposal, John Wiley & Sons, Inc., New York, N.Y. (1954).

2. Goodman, W.I., and Freund, E.C., Principles and Practices of Urban Planning, International City Manager's Association, Washington, D.C. (1968).

3. Sewage Treatment Plant Design, Water Pollution Control Federation Manual of Practice No. 8 (1963).

4. Design and Construction of Sanitary and Storm Sewers, Water Pollution Control Federation Manual of Practice No. 9 (1963).

5. Smith, R., and Eilers, R.G., "Cost to the Consumer of Collecting and Treating Wastewater in the United States," in press, R.A. Taft, Water Research Center, Cincinnati, Ohio.

CHAPTER 3

FLOW EQUALIZATION

3.1 General

The stream of wastewater entering most treatment plants varies significantly throughout the day in rate of flow and contaminant concentration. This is caused by the habits of the population served and the hydraulic characteristics of the sewerage system. The extent of this variation is shown in References (1), (2) and (3) which report measurements made on operating plants. As a result of this variation in the plant influent stream, the quality of the effluent streams from processes also varies.

Measurement of peak flows versus 24-hour average flows in trunk sewers from residential communities in Orange County, California, was reported in a study by Boyle Engineering and Lowry and Associates (4). The data fitted the following relationship:

$$\text{Peak Flow, MGD} = 1.84\,(\text{Average Flow, MGD})^{0.92}$$

The peak to average ratio thus decreases as the average flow increases.

Generally, the design of almost all of the major structural elements comprising a treatment plant is based on the average rate of flow. Other elements of the plant, such as conduits, siphons and distributing mechanisms are designed on the basis of the extreme peak rate of flow which is usually considered as three times the average rate of flow. Older plants, which are supplied by combined sewer systems, must also take into account the effect of heavy rainfall and storm flows. Modern collection systems utilize separate sanitary and storm sewers, but must still consider contributions from ground water infiltration, illegal tie-ins and extraneous surface water that may enter the collection systems. Similarly, consideration of the minimum rate of flow is necessary in the design of certain elements of the total plant, such as grit chambers, measuring devices and dosing equipment. For this purpose, the average rate of flow for the 4-hour minimum flow period is used. This is often assumed to be 40 percent of the average hourly flow rate. Under normal conditions, the design of individual treatment units should be based on the average hourly flow rate of domestic sewage plus the average hourly rate of flow of industrial wastes during the maximum significant period of the day. Where recirculation is employed through any individual unit, or groups of units, the recirculated effluent must be added to the flow rate originally considered.

Flow equalization, although commonly used in industrial waste treatment practice, has been generally neglected in traditional municipal wastewater treatment facilities. There are two major objectives in the design of flow equalization basins. The first of these is simply to dampen the diurnal flow variation that normally exists in typical municipal wastewater collection systems. This is done to achieve a constant or nearly constant flow rate through the subsequent treatment processes. In this type of system, little consideration is given to controlling the quality changes that take place during storage. The major design factors are to supply sufficient air to keep the basin aerobic and to provide adequate turbulence to prevent solids deposition.

The second objective of flow equalization is to provide the capacity to distribute shock loads of toxic or process-inhibiting substances over a reasonable period of time so as to prevent system failure and to minimize the adverse effects of periodic discharges of harmful contaminants to the receiving stream or surface impoundment. Time-dependent concentration profiles and flow-through curves are normally used to analyze the flow characteristics of these systems and to determine the effect of tank geometry, placement of effluent weirs and mixing regime on changes in contaminant concentrations through the basin.

Detailed discussions of the flow equalization design principles can be found elsewhere (2,3). In essence, the justification for flow equalizing facilities is economic, although other factors, such as solids settling and septicity, must be considered. The economic justification involves the relative costs of providing equalization facilities as opposed to increasing design capacities of subsequent unit processes of the waste treatment facility to produce an effluent of equal quality. Because of the different degrees of sensitivity inherent in each type of treatment process, the economic balance is unique to each installation. Other factors which affect such an analysis are related to esthetics, land values, areal limitations, and the economics of size for each of the unit processes.

A short discussion of the effects of flow variation on several unit processes is presented in the following sections.

3.2 Clarifiers

In general, the clarifier is usually of sufficient capacity to serve as a buffer against small surges in flow. If, however, the clarifier influent flow is unusually large, exceeding the unit's design capacity, an overflow line from the parimary clarifier discharging to the influent wet well or, ideally, a flow equalization basin may be necessary. However, if the flow is lower than design capacity, the waste will be held longer with possibly improved clarification. Since the overriding influence on clarifier efficiency is overflow rate, solids removal will be affected by flow variations. If the basin is properly designed, these effects are minimal. The allowance of excess capacity in the design provides an additional safeguard.

3.3 High-Rate Settlers

Tube settlers are only slightly affected by variations in flow. At times of extreme high flow, the detention time will be markedly decreased. At periods of low flow, the settling time will be lengthened with some improvement in solids removal.

3.4 Microscreens

A microscreen unit is designed to maintain a constant level differential across the unit by automatically increasing the drum speed when the debris concentration or the flow rate increases and decreasing the drum speed when the debris concentration or flow rate decreases. Therefore, within limits, the microscreen is unaffected by change in flow rate.

3.5 Filters

Flow variation can have a little or a great effect on the operation of a conventional, multi-media or up-flow filter, depending on the nature of the solids in the influent stream. Usually, filters are operated at a constant rate of flow and should be protected from the flow variations which could damage product quality. Flow equalization ahead of this process should be provided.

3.6 Moving Bed Filter

The units are installed with level controls in the head tank which increase or decrease the sand pulse rate in direct proportion as the volume of the liquid feed changes. Similarly, in the event of extreme low flow, the units are automatically shut down.

3.7 Flotation

Flotation, like sedimentation, is primarily responsive to the overflow rate. It generally requires a relatively constant flow rate for proper performance. Flow equalization should be provided ahead of these units.

3.8 Solids - Contact Units

Solids-contact clarifiers are very sensitive to flow changes. To counteract this sensitivity, sludge blankets must be kept low to allow for upsets. Sludge blanket control devices are desirable.

3.9 Summary

If space is available, a flow equalization basin should be constructed when a tertiary treatment system is added to an existing secondary plant, as the latter may be frequently upset. Preferably, the flow equalization should be incorporated into the system. If space is not available for a holding pond, variable speed pumps whose speed is controlled by the wet well level can be used to deliver the secondary effluent to the tertiary treatment plant.

3.10 References

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CHAPTER 4

COAGULATION OF WASTEWATER

4.1 General

Wastewaters contain a wide variety of organic and inorganic suspended solids. These solids may have been present in the raw wastewater or may have been precipitated from solution during previous stages of treatment. All must be removed if a high quality effluent is to be produced.

A number of factors will influence the rates at which wastewater solids may be removed by sedimentation. Particularly important is particle size. Small particles in the colloidal range will not settle out in a practical detention time and must be agglomerated into larger particles which will settle at reasonable rates.

Small particles have very large surface areas per unit weight of solid with associated forces which tend to keep the particles separated or in a stable state. Effecting aggregation of the solids is a matter of overcoming these stabilizing forces by application of selected chemical coagulants. It is not the purpose of this manual to cover the subject of coagulation in detail. Only such information which is useful in process control is provided. More extensive coverage can be found in a number of publications for those who desire further information (1),(2), (3).

The principle stabilizing forces are electrostatic repulsion from electrical double layers surrounding particles suspended in water and physical separation of particles by films of water adsorbed on the particle surfaces. The electrical double layer is due to an imbalance of ions near the particle surface and imparts an electrical charge to particles which is generally negative for domestic wastewater particles (4). The particles with like charges then tend to repel one another. The magnitude of the charge on the double layer must be reduced to permit the particles to come together for agglomeration. Reduction of the charge is a principle function of the coagulant.

Principal natural destabilizing or aggregating forces are Brownian movement and Van der Waals forces. Brownian movement is a constant random motion of small particles due to collisions of the particles with thermally-agitated water molecules, while Van der Waals forces are atomic dipole interactions which exist between all atoms. Van der Waals forces, which are always attractive, predominate at short distances from the particle surfaces. If Brownian movement causes two or more particles to approach within a short distance from each other, and if the repulsive electrical forces have been sufficiently reduced, then the particles will be held together by the Van der Waal forces. Other particles can be added to the particle pair until the resulting aggregate or floc reaches such a size that it rapidly settles out of solution.

These natural destabilizing forces are generally not sufficiently rapid to permit effective solids removal from wastewater, but must be augmented by addition of chemical coagulants to destabilize and tie the particles together and by application of hydraulically-or mechanically-applied mixing to promote rapid collisions of the destabilized particles. A number of mechanisms have been proposed to explain the action of coagulants in promoting particle aggregation. These include charge reduction, physical or chemical bridging of coagulant molecular chains between particles and physical enmeshment of particles in a mass of precipitated coagulant. All of these mechanisms probably play a greater or lesser role in effective coagulation.

Coagulants used in wastewater treatment include those used in potable water treatment with some additions. These are: alum, sodium aluminate, ferric chloride, ferric sulfate, ferrous chloride, ferrous sulfate, lime and organic polyelectrolytes. Other materials such as soda ash or clays may be used as sources of alkalinity or weighting agents, respectively, to aid coagulation. When added to water, salts of aluminum or iron react with the water or alkalinity present in the water to form insoluble hydrolysis products. It is these hydrolysis products which are the effective coagulating agents. These materials which are positively charged in the neutral pH range adsorb on the negatively charged wastewater particles reducing repulsive forces between the particles. The coagulant may also react with other constituents of the wastewater, particularly anions such as phosphate and sulfate, forming hydrolysis products containing various mixtures of ions. These various hydrolysis products differ in their effectiveness as coagulants. The chemistry of the reactions is extremely complex.

For each combination of coagulant and wastewater there is an optimum dosage of coagulant and an optimum pH range for coagulation. These are the two parameters which are generally controlled in operation of advanced waste treatment plants for solids removal through coagulation.

4.2 Coagulation Control

Because coagulation represents a group of complex reactions, laboratory experimentation is essential to establish and maintain the optimum coagulant dosage and the effect of important variables on the quality of coagulation of the wastewater under investigation. With hydrolyzing coagulants three procedures may be followed for this purpose: the jar test, measurement of zeta potential, and measurement of phosphate content of the wastewater. Lime is a special case. Proper control of lime addition may usually be maintained by measuring the pH or automatically titrating alkalinity after lime addition.

4.2.1 Jar Test

The single, most widely used test to determine dosage and other parameters is the jar test. The equipment for this test and the directions for its proper performance have been published(5),(6),(7),(8). The jar test attempts to simulate the full scale coagulation-flocculation process and has remained the most common control test in the laboratory since its introduction in 1918. Since the intent is to simulate an individual plant's conditions, it is not surprising that there has been no standardization of the test. The jar test as variously performed does, however, have some elements of conformity. In its essentials, the jar test simply consists of a series of sample containers, usually six, the contents of which can be stirred by individual mechanically-operated stirrers. Water to be treated is placed in the containers and treatment chemicals are added while the contents are being stirred. The range of conditions, for example, coagulant dosages and pH, are selected to bracket the anticipated optima. After a short, 1-5 minute, period of rapid stirring to ensure complete dispersion of coagulant, the stirring rate is decreased and flocculation is allowed to continue for a variable period, 10 to 30 minutes or more, depending on the simulation. The stirring is then stopped and the floc are allowed to settle for a selected time. The supernatant is then analyzed for a variety of parameters. With wastewater the usual analyses are for turbidity or suspended solids, pH, residual phosphorus and residual coagulant.

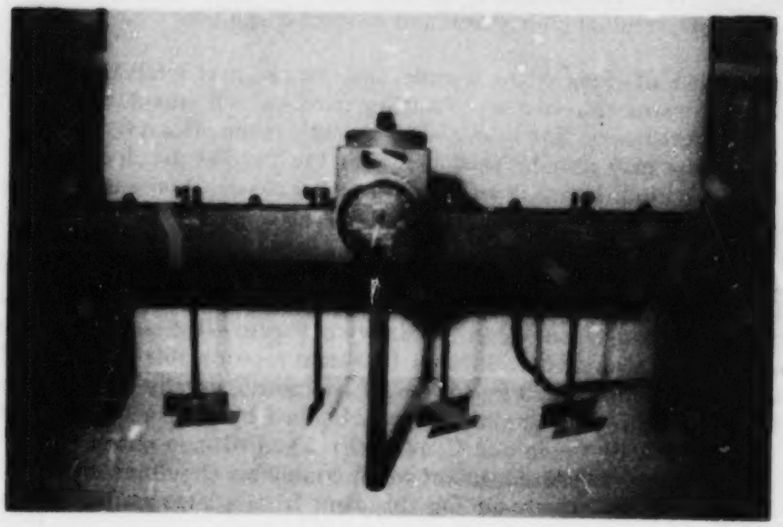
If desired, a number of supernatant samples may be taken at intervals during the settling period to permit construction of a set of settling curves which provide more information on the settling characteristics of floc than a single sample taken after a fixed settling period. A dynamic settling test may also be used in which the paddles are operated at 2 to 5 rpm during the settling period. This type of operation more closely represents settling conditions in a large horizontal basin with continuous flow.

Several six-position stirrers are available commercially for running jar tests; one from Phipps and Bird, (Phipps and Bird, Inc., Richmond, Va.),another from Coffman Industries, (Coffman Industries Inc., Kansas City, Ka.), are shown in Figure 4-1. Standard laboratory mixers have also been used; however, it is difficult to obtain reproducible mixing conditions using different pieces of equipment. Various types of containers, usually beakers or jars, are used to hold the samples. Improved mixing may be obtained by adding stationary plates in the containers as described by Camp and Conklin (8). The Coffman stirrer has an attachment which makes it possible to add coagulant to all containers simultaneously, however, good results can be obtained by rapidly adding coagulant from a large graduated pipette to each jar in sequence.

A simple apparatus, shown in Figure 4-2, can be constructed from tubing, rubber stoppers and small aquarium valves to permit rapid sampling of supernatant. The unit is placed next to the sample jars at the beginning of the settling period with the curved stainless steel tubes dipping into the jars. At desired intervals the vent valve is covered with a finger, permitting vacuum to draw samples into the small sample bottles. The needle valves are adjusted so that supernatant is drawn into all the bottles at the same rate. When sufficient sample is obtained, the vent is uncovered and the bottles are replaced with empties. The maximum sampling rate is about once per minute.

Figure 4-3 shows typical types of settling curves which may be obtained. Curve A indicates a coagulation which produced a uniformly fine floc, so small that at the end of 1 to 2 minutes settling, the supernatant had a turbidity equal to that of the starting water due, in part, to the fine floc which resisted settling. Settling was slow and the final turbidity was excessive. This coagulation would not be satisfactory in an advanced wastewater treatment plant. Curve B represents the most common type of settling rate obtained. During the first 5 minutes, the settling rate was practically a straight line on a semilog plot. Settling was rapid and clarification was satisfactory. The coagulation represented by curve C shows that a mixture of large rapid settling floc and small, slow-settling particles was obtained. Settling was rapid for the first two minutes, but with little further clarification after that. High residual turbidity may also have resulted from incomplete coagulation. Curve D represents the ultimate in coagulation. Practically all of the floc particles were so large and dense that 97% settled within three minutes. Sedimentation was essentially complete within that time since only 0.5% additional floc settled in the next 27 minutes. Final clarity of the supernatant was entirely satisfactory. This coagulation was obtained with a coagulant aid.

Measurement of turbidity provides the most rapid indication of the degree of solids removal obtained. The recommended procedure for turbidity measurement by light scattering is given in the 13th edition of *Standard Methods for Examination of Water and Wastewater;* however, other methods varying from simple visual evaluation to measurement of light



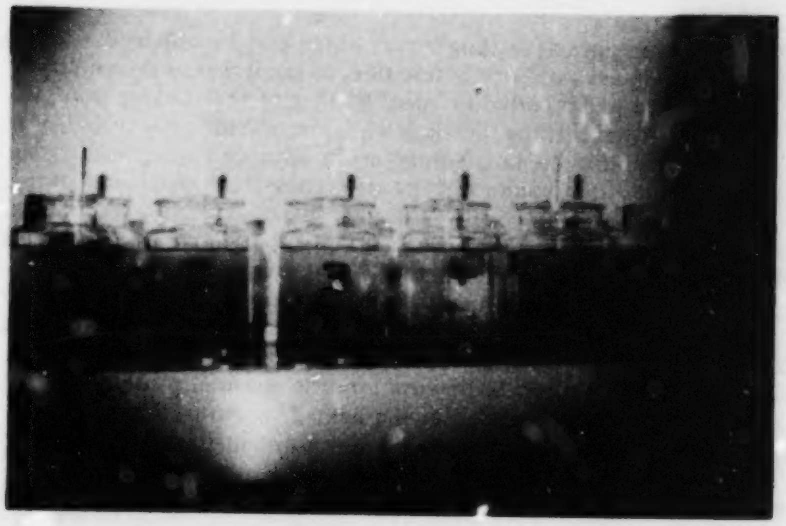
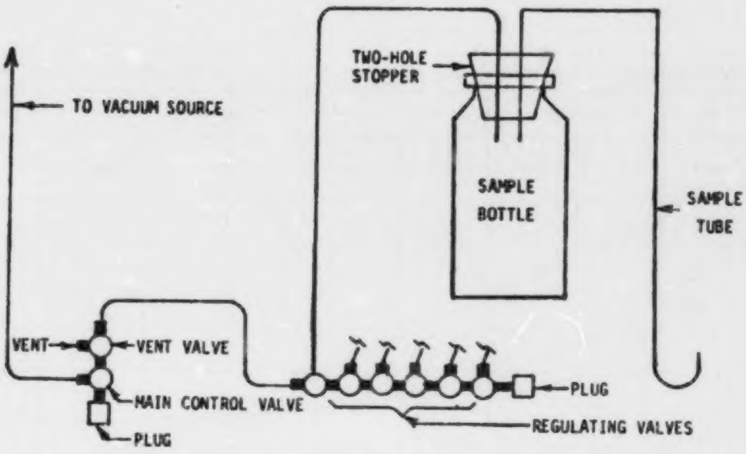


Figure 4-1. JAR TEST UNITS WITH MECHANICAL (TOP) AND MAGNETIC (BOTTOM) STIRRERS


TWO-HOLE STOPPER
TO VACUUM SOURCE
SAMPLE BOTTLE
SAMPLE TUBE
VENT
VENT VALVE
MAIN CONTROL VALVE
PLUG
PLUG
REGULATING VALVES

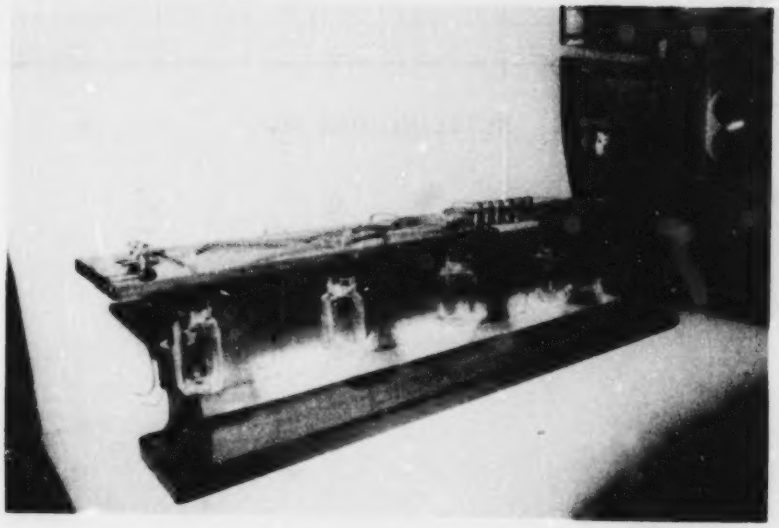


Figure 4-2. SIX-POSITION SAMPLER

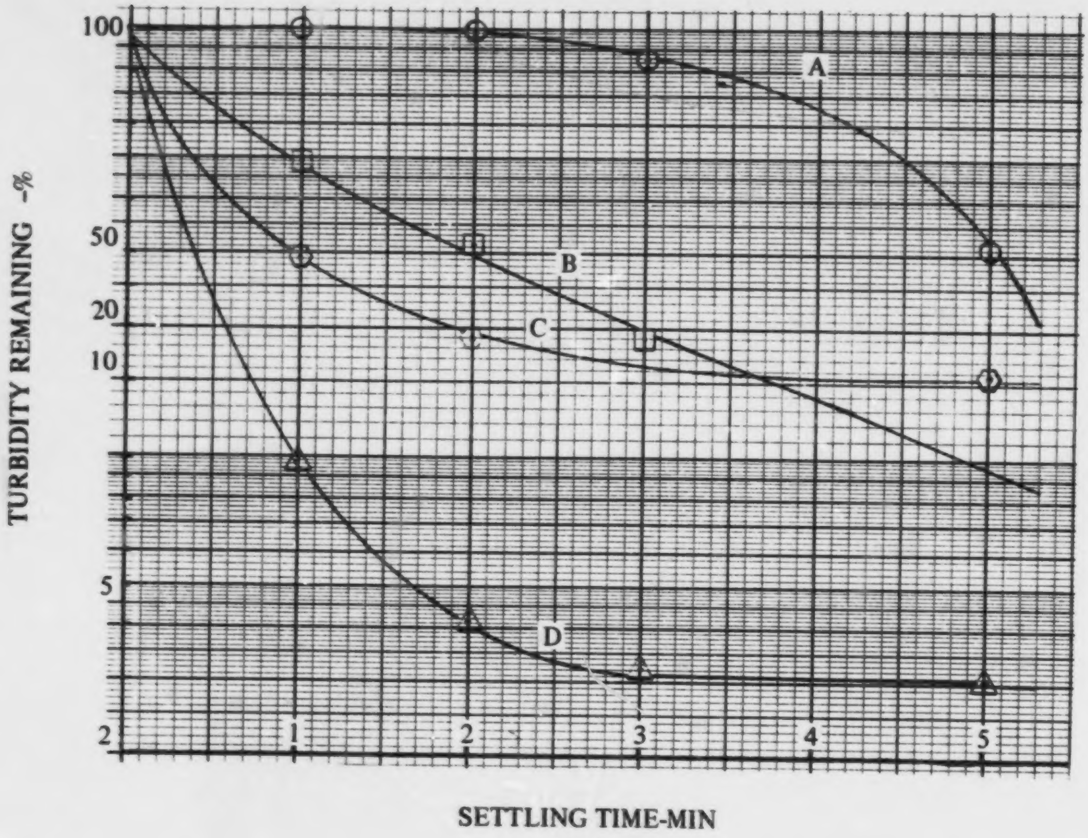


Figure 4-3. SETTLING CURVES FREQUENTLY OBTAINED

transmitted on a laboratory spectrophotometer can be used for purposes of comparison. Measurement of residual suspended solids is the only procedure which gives the actual weight concentration of solids remaining, but the procedure is too slow for purposes of process control. Residuals of phosphate and coagulant in supernatant are usually of interest and may be measured either manually or with automated equipment.

A typical jar test might be run as follows: Wastewater samples are placed in containers and rapid mix is started at 100 rpm. Selected dosages of coagulant are rapidly added to the containers covering the expected range of the optimum dosage and a timer is started. If a polymer is to be used as a coagulant aid, it is added to each jar after two minutes and rapid mixing is continued for one additional minute. The paddles are then slowed to 30 rpm and mixing continues for 10 minutes. The paddles are then stopped and the sampling apparatus previously-described is placed in position. At settling times of 1, 3, 5, 10 and 20 minutes samples of supernatant are drawn for turbidity measurement. After the final turbidity sample is drawn, a larger volume of supernatant may be decanted for more complete analysis. Results are plotted as in Figure 4-4 for judgment as to the desired coagulant dosage. The jar test may be repeated using a smaller dosage range around the observed optimum to more closely locate the best dosage.

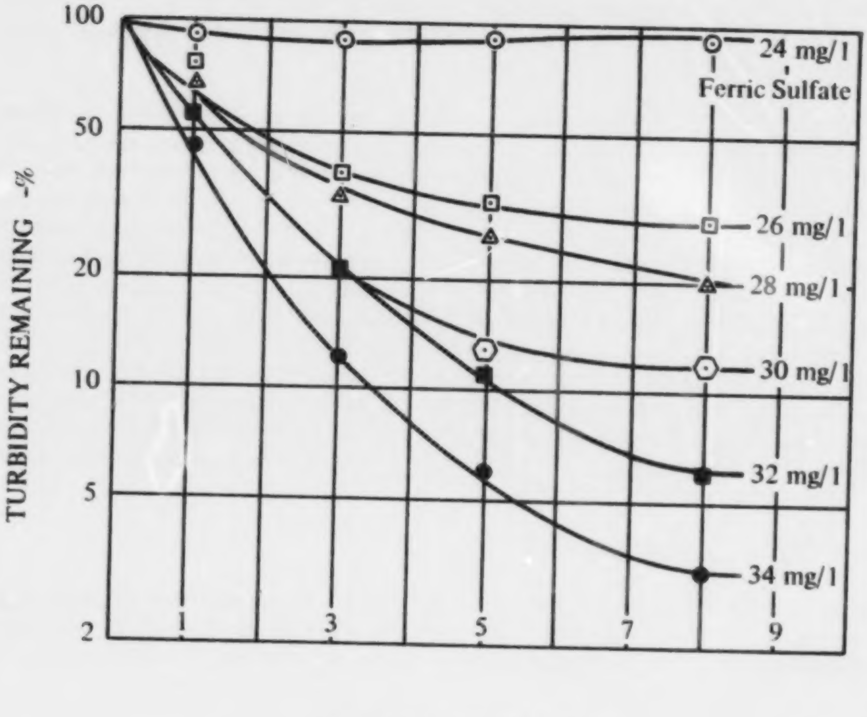


Figure 4-4. JAR TEST RESULTS

If additional alkalinity is required to hold the coagulation in the optimum pH range, this should be added to the samples ahead of the coagulant. Once an approximate optimum dosage has been determined, it may be desirable to repeat the jar test with the optimum coagulant dosage but using varying quantities of added alkalinity to give different pH values. Experience in coagulating a given wastewater provides the best guide as to the best methods for controlling the process.

4.2.2 Zeta Potential

Measurement of particle charge is another procedure which may be useful for control of the coagulation process(9),(10),(11). The total particle charge is distributed over two concentric layers of water surrounding the particle; an inner layer of water and ions which is tightly bound to the particle and moves with it through the solution and an outer layer which is a part of the bulk water phase and moves independently of the particle. The surface charge is not measurable experimentally; however, the zeta potential which is the residual charge at the interface between the layer of bound water and the mobile water phase can be determined with a commercially-available instrument.*

In the zeta potential measurement procedure, a sample of treated water containing floc is placed in a special plastic cell under a microscope as shown in Figure 4-5. Under the influence of a voltage applied to electrodes at the ends of the cell, the charged particles will migrate to the electrode having a polarity opposite that of the particle. The velocity of migration will be proportional to the particle charge and to the applied voltage. The particle velocity can be calculated by observing the time it takes a particle to travel a given distance across an ocular micrometer. The zeta potential can then be obtained from a chart which combines the particle velocity with instrumental parameters. Detailed operating instructions are supplied with the instrument.

To control the coagulation by zeta potential, samples of water while being mixed are dosed with different concentrations of coagulant. Zeta potentials are then measured and recorded for floc in each sample. The dosage which produces the desired zeta potential value is applied to the treatment plant. Zeta potentials of floc produced in the plant may also be measured as a means of control. The precise zeta potential which signals optimum coagulation must be determined for a given wastewater by actual correlation with jar tests or with plant performance as in Figure 4-6. The control point is generally in the range of 0 to -10 millivolts. If good correlations can be obtained between some zeta potential values and optimum plant performance, then it is possible to make rapid measurements of particle charge to compensate for major variations in wastewater composition due to storm flows or other causes. Short term variations such as those due to sudden industrial waste dumps are still beyond control with any present techniques because of the time lag between recognition of a problem with coagulation and adoption of a satisfactory change of coagulation conditions.

4.2.3 Phosphate Monitoring

A third means of coagulant dosage control where the coagulant is being used to precipitate phosphate as well as to remove solids is to automatically analyze the incoming wastewater for soluble orthophosphate. The coagulant is then paced to maintain a selected ratio of

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**A product of Zeta-Meter, Inc., N.Y., N.Y.*

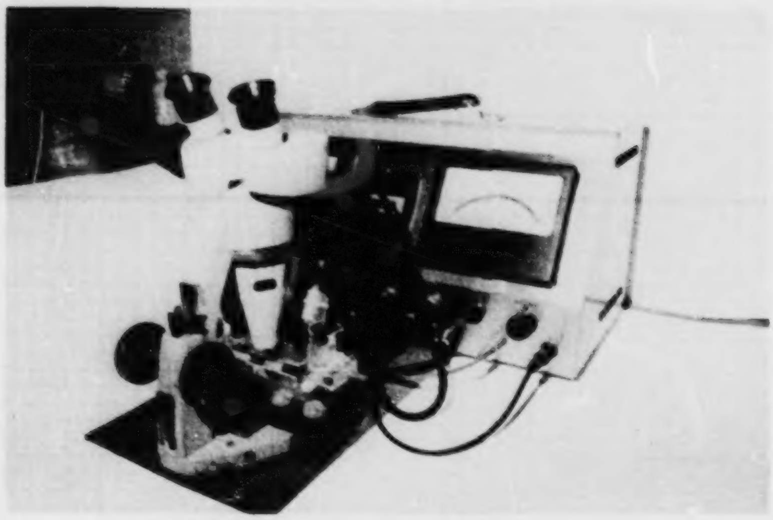

Figure 4-5. ZETA POTENTIAL APPARATUS

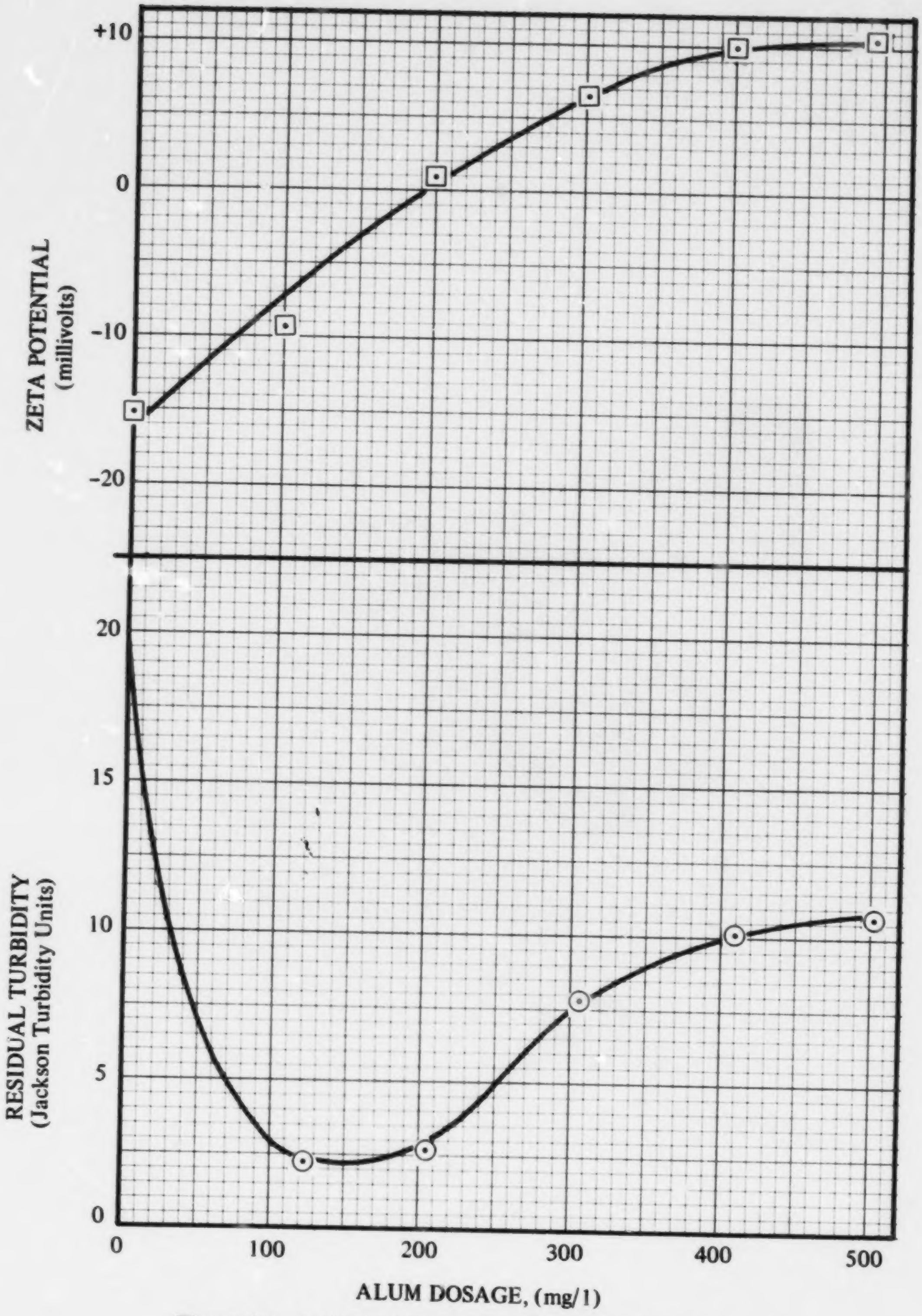

Figure 4-6. COAGULATION OF RAW SEWAGE WITH ALUM

coagulant to phosphate either automatically or by frequent manual adjustment. The Technicon Auto-Analyzer* which is commercially available has been adapted for this purpose. Dow Chemical Co. has developed an automatic system which will add ferric chloride in proportion to soluble orthophosphate with corrections for varying flow and concentration of ferric chloride.

Coagulant dosages which produce good phosphate removal will also generally result in good solids removal if polymers are used to aid in flocculating the fine precipitated matter. Under normal conditions, the polymer feed rate should be varied with flow to maintain a constant dosage; however, the polymer feed rate does not have to be changed with changes in dosage of the inorganic coagulant.

**A product of Technicon Corporation, Tarrytown, N.Y.*

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CHAPTER 5

CHEMICALS AND FEEDING

5.1 General

This chapter surveys the various available chemicals that are used for the purpose of solids removal and provides pertinent facts concerning their handling, transport and storage. The composition and physical properties of these chemicals are discussed, and a short review of polymeric flocculant aids and their properties is included. A review of the various chemical feeders and their application to specific coagulants is also included.

5.2 Chemicals for Coagulation and pH Control of Wastewater

5.2.1 Aluminum Sulfate

Ordinary ground alum is a mixture of rice-size material and fines. This grade is used by the majority of water and wastewater plants. In general, ground alum is easy to feed and does not bulk in hoppers. It is non-corrosive, requiring no protection for the hopper interiors. (Specifications for filter alum may be obtained from the Amer. Wat. Wks. Assn., 521 Fifth Avenue, N.Y., N.Y. 10017.)

Rice grade alum is ground alum from which the fine material has been screened. Powdered alum is the very fine material that is screened out to obtain the rice grade. This grade will bulk in hoppers and can be extremely dusty and difficult to feed. Powdered alum is seldom, if ever, used by wastewater plants. Characteristics of the several forms of dry alum are given in Table 5-1.

Table 5-1

CHARACTERISTICS OF ALUM (1)

Approx. Composition		$Al_2(SO_4)_3 \cdot 14 H_2O$
Al_2O_3 Content		17% (Min.)
Fe_2O_3		0.75% (Max.)
Insolubles		0.5% (Max.)
Hygroscopic		Very Slightly
Form	Lump, ground, rice powder	
Color		Ivory White
Weight per cu ft		
Lump	60-70 lbs.	
Ground	63-76 lbs.	
Rice	52-62 lbs.	
Powdered	38-45 lbs.	

Table 5-1 (Continued)

Angle of Repose (Approx.)	
Ground	38-45°
Rice	33-38°
Powder	65°
Solubility	
Temp.	Lb Alum (17% Al_2O_3) Per Gal. of Water
32°F	7.9
60°F	8.4
100°F	9.1
pH of Solution (1%)	3.4

AWWA Specifications for "purified" alum - Commercial "purified" alums vary from 16.8 to 17.5% Al_2O_3. For "unpurified" filter alum, AWWA specifications allow 10% insolubles. This type alum, sold under the trade name of "Activated Alum", contains 10 to 12 molecules of water of hydration, approx. 18.5% Al_2O_3, 0.3% Fe_2O_3, and 9% insolubles.

Lump alum ranges in size from 3/4 to 3"; ground alum passes 100% thru 4 mesh and 95% thru 10 mesh; rice alum passes 100% thru 6 mesh and 5% thru 20 mesh; powdered alum passes 95% thru 100 mesh.

a) Handling and Storage

Commercial filter alum is usually shipped in 100 lb multi-wall paper bags, or in bulk, in box cars or hopper cars for unloading by pneumatic or mechanical equipment.

Filter alum will cake when stored in bags in damp places, but it is usually free-flowing when stored in dry concrete or steel bins. Storage bins are usually built with 60° slopes to the hopper bottoms to insure complete emptying and to avoid arching.

Filter alum should not be handled in mechanical equipment that has close-fitting rotating parts; it has the peculiarity of building up a flint-like coating due to the pressure and sliding action of close fitting surfaces. (For instance, star feeders having rotors that fit the housing too closely will eventually build up such resistance to turning that shafts will be sheared or motors stalled.)

b) Materials for Handling

Dissolving tanks, piping and pumps may be made of Duriron, Durimet-20, hard or soft rubber, plastic, ceramic or lead. Asphalt and cypress lined dissolvers with acid bronze outlets and Everdur are satisfactory. Dry feeders may be of standard construction, but any portion of the equipment in contact with solutions of alum must be corrosion resistant.

c) Use of Alum

In treatment plants, alum is used as a coagulant to remove suspended solids. Reactions between alum and the natural constituents of various wastewaters are influenced by many factors, hence it is impossible to predict accurately the amount of alum that will react with a given amount of natural alkalinity, lime or soda ash which may have been added to the wastewater. Theoretical reactions can be written which will serve as a general guide, but, in general, the optimum dosage in each case must be determined by laboratory jar tests.

The simplest case is the reaction of Al^{+++} with OH^- ions made available by the ionization of water or by the alkalinity of the water.

Solution of alum in water produces:

$$Al_2(SO_4)_3 \rightarrow 2\,Al^{3+} + 3\,SO_4^{2-}$$

Hydroxyl ions become available from ionization of water:

$$H_2O \rightarrow H^+ + OH^-$$

The aluminum ions (Al^{3+}) then react:

$$2\,Al^{3+} + 6\,OH^- \rightarrow 2\,Al(OH)_3\downarrow$$

Consumption of hydroxyl ions in the water will result in a decrease in the alkalinity. Where the natural alkalinity of the water is inadequate for the alum dosage, the pH must be increased by the addition of hydrated lime, soda ash, or caustic soda. The reactions of alum with the common alkaline reagents are shown in Table 5-2. While these reactions are an oversimplification of what actually takes place, they do serve to indicate orders of magnitude and to point out that sulfate ion and CO_2 are byproducts of alum treatment.

Table 5-2

REACTIONS OF ALUMINUM SULFATE

$$Al_2(SO_4)_3 + 3\,Ca(HCO_3)_2 \longrightarrow 2\,Al(OH)_3\downarrow + 3\,CaSO_4 + 6\,CO_2\uparrow$$

$$Al_2(SO_4)_3 + 6\,NaHCO_3 \longrightarrow 2\,Al(OH)_3\downarrow + 3\,Na_2SO_4 + 6\,CO_2\uparrow$$

$$Al_2(SO_4)_3 + 3\,Na_2CO_3 + 3\,H_2O \longrightarrow 2\,Al(OH)_3\downarrow + 3\,CO_2\uparrow$$

$$Al_2(SO_4)_3 + 6\,NaOH \longrightarrow 2\,Al(OH)_3\downarrow + 3\,Na_2SO_4$$

$$Al_2(SO_4)_3 + 3\,Ca(OH)_3 \longrightarrow 2\,Al(OH)_3\downarrow + 3\,CaSO_4$$

In terms of quantities, the reactions in Table 5-2 can be expressed as follows:

1 mg/l of alum reacts with:

- 0.50 mg/l natural alkalinity, expressed as $CaCO_3$
- 0.33 mg/l 85% quicklime as CaO
- 0.39 mg/l 95% hydrated lime as $Ca(OH)_2$
- 0.54 mg/l soda ash as Na_2CO_3

These approximate amounts of alkali when added to water will maintain the alkalinity of the water unchanged when 1 mg/l of alum is added. For example, if no alkalinity is added, 1 mg/l of alum will reduce the natural alkalinity by 0.50 mg/l as $CaCO_3$, but alkalinity can be maintained unchanged if 0.39 mg/l of hydrated lime is added. This lowering of natural alkalinity is desirable in many cases to attain the pH range for optimum coagulation.

For each mg/l of alum dosage, the sulfate (SO_4) content of the water will be increased approximately 0.49 mg/l and the CO_2 content of the water will be increased approximately 0.44 mg/l. The general pH range, wherein the best coagulation occurs, is from pH 5.5 to 8.5. However, for a given wastewater, the optimum pH will likely fall within a relatively narrow portion of this broad range. In tests with pure solutions, phosphate precipitation with alum was optimum at a pH of 6.0 (2), but in wastewater alum is effective for phosphate removal over a wide pH range. A good summary of available data on alum coagulation can be found in an article by Farrell, et al. (3).

d) Safety

Alum is irritating to the skin and mucous membranes because of its acidic nature. When handling dry alum, be especially careful to protect against dust, as it can cause serious eye injury. Wear a cap and bandana, dust mask, close-fitting goggles, and gloves. Wear loose, denim quality, dustproof, long sleeve clothing. Tie trouser cuffs and sleeves to prevent dust from entering. Exposed skin surfaces should be coated with a protective cream.

5.2.2 Liquid Alum

Liquid alum is aluminum sulfate in solution. One gallon of liquid alum weighs approximately 11 pounds and contains the equivalent of 5.4 pounds of dry aluminum sulfate. Some water treatment plants prefer liquid alum because it is more easily fed.

All handling procedures for dry alum apply to liquid alum (4). In addition, the use of face shields or fitted goggles, boots, gloves and a rubber apron to protect the operator from splashes or sprays is recommended. Use of liquid alum is usually restricted to facilities located within a radius of approximately 50-75 miles from the supplier, due to the dead weight of solution water which increases shipping costs.

a) Characteristics

The strengths of the usual grades of liquid alum are as follows:

Deg. Baumé	Al_2O_3
27.0°	5.8%
32.2°	7.2%
36.4°	8.3%

Liquid alum (32.2° Baumé) has a specific gravity of 1.285 at 60°F and weighs 10.7 lbs per gal. Commercial liquid alum is a true solution containing less than 0.2 percent insolubles. Storage at lower than freezing temperatures may cause crystal formation.

Since dry commercial filter alum contains at least 17 percent Al_2O_3 (AWWA Specifications), then 32.2° Baumé liquid alum containing 7.2 percent Al_2O_3 is 7.2 ÷ 17 or 42.3 percent of the strength of dry filter alum on a weight basis.

b) Handling and Storage

Customers' storage tanks should have a capacity at least double that of delivery tank trucks (2,000 to 4,000 gal) or tank cars (6,000 to 8,000 gal). There is no deterioration or loss by storage of liquid alum.

Tank trucks are equipped with two-inch centrifugal pumps and rubber hose for pumping to customers' storage. Unloading time may vary from 20 min to 2 hr depending upon the pumping head. Tank cars are unloaded by air or gravity with equipment installed at receiving stations.

Liquid alum should be transferred from storage to feeding equipment by centrifugal pumps.

c) Materials for Handling

Storage tanks for all strengths may be lead-lined steel, lead-lined Douglas Fir or rubber-lined steel. Wooden tanks are suitable for weak solutions, but not for strong solutions because of delignification of the wood.

Pumps may be Worthite, Duriron, Durimet 20, Pioneer Metal, Aloyco 20 or other nickel-chrome alloys.

Pipe should be lead-lined or rubber-lined steel, flanged and 125 lb standard test. Pipe fittings should be lead or standard cast iron with lead or rubber lining, flanged and 125 lb standard test. Gaskets should be rubber or rubber impregnated asbestos or graphited asbestos. Rubber hose is also very satisfactory for conveying alum solutions.

d) Use of Liquid Alum

To convert AWWA Specification dry filter alum dosage to liquid alum dosage, multiply dry alum requirements by the following factors, which are obtained by dividing the Al_2O_3 content of AWWA Specification filter alum (17 percent) by the Al_2O_3 content of the three grades of liquid alum (5.8, 7.2 and 8.3 percent):

Strength °Baumé	Factor
27.0°	2.93
32.2°	2.36
36.4°	2.05

5.2.3 Sodium Aluminate

Sodium aluminate ($Na_2Al_2O_4$) is an alkaline material. Most of the experience with aluminate has been with its addition to the aeration tank of an activated sludge system to precipitate phosphate. In these cases, the alkaline nature of the salt is neutralized by the carbon dioxide produced by the organisms. If aluminate is used at some other point in the process, it must be determined that the pH of the wastewater is not adversely affected.

As in the case of alum, the usual dose of aluminate, on an aluminum basis, is one or two times the phosphorus content of the wastewater. Aluminate is available in both dry powder (32.91% Al) and liquid (2.27 lb Al/gal) forms.

Because of its caustic nature, both dry dust and liquid forms of aluminate are irritating to the skin, eyes and mucous membranes. The dry powder must be stored in an area free of water or excessive moisture. The shelf life is about six months, and the liquid form should be protected from carbon dioxide absorption during storage.

a) Handling

Materials used for handling dry or liquid aluminate must be resistant to alkali. Black iron or stainless steel are recommended. Materials such as brass, bronze, copper and aluminum cannot be used. Neoprene hose can be used for transfer lines. Liquid aluminate is a very viscous material and positive displacement pumps should be used for controlling dosage. At low temperatures heating of the liquid aluminate to 70°F may be necessary for proper flow characteristics.

b) Calculation of Dosage

To determine the amount of aluminate to be used, there are three operations to be accomplished: (1) determine the total daily amount of phosphorus, (2) the equivalent weight of aluminate, and (3) adjust for available aluminum. A 10-million-gallon-per-day treatment plant flow containing 10 milligrams of phosphorus per liter of sewage, measured as elemental phosphorus and employing a dose of one milligram of aluminum per milligram of phosphorus, can be used as an example. This would require 830 pounds of aluminum which would be contained in 3,450 pounds of dry aluminate. If liquid aluminate were used, 500 gallons per day would be required.

5.2.4 Ferric Sulfate

Commercial ferric sulfate [$Fe_2(SO_4)_3 \cdot X\ H_2O$] is marketed in 100 lb moisture-proofed paper bags, 400 lb fibre drums, and in bulk in steel hopper cars (5). The general characteristics of two commercial forms are shown in Table 5-3.

Table 5-3

CHARACTERISTICS OF FERRIC SULFATE

Trade Name	"Ferri-floc"	"Ferriclear"
Approx. Composition	$Fe_2(SO_4)_3 \cdot 3H_2O$	$Fe_2(SO_4)_3 \cdot 2H_2O$
$Fe_2(SO_4)_3$ Content	68%	76% min
Fe^{+++} Content	18.5%	21%
Fe^{++} Content	3.0% max*	3.3%
Alumina	1.0%	(?)
Water Insolubles	Yes	Yes
Weight per cu ft	60 – 74 lb	78 – 90 lb
Color	Reddish-gray	Grayish-white
Hygroscopic	Slightly	Slightly
Form	Granular	Granular
*Usually less than 1%		

The partially hydrated forms are only slightly hygroscopic and thus are generally easy to transport, feed and dissolve. They do, however, tend to cake at higher relative humidities. When stored in bulk concrete storage bins, a thin shell or crust may form over the surface of the material in the bin, thus effectively sealing out moist air from the rest of the material.

When these chemicals are shipped in bulk, they are usually handled by mechanical conveying equipment such as screw conveyors, bucket elevators or bulk-flo type conveyors. Air conveying is practiced, but requires increased equipment maintenance, especially in damp climates. These chemicals are not corrosive when dry, but are actively corrosive in solution.

When handling these chemicals with air conveyors used for other chemicals, the system should be carefully cleaned and the filter bags changed after each use. If the same conveying equipment is used for both quicklime and ferric sulfate, precautions should be taken against possible mixing of the two, because a mixture will heat and cake.

a) Materials for Handling

Dissolvers, piping and pumps may be type 316 stainless steel, Duriron, hard or soft rubber, plastic, ceramic or lead. The dry chemicals can be stored safely in ordinary steel bins of chemical feeder hoppers.

Dry feeders may be of standard construction generally, but parts in contact with the solution, vapor or fumes should be of stainless steel.

b) Use of Ferric Sulfate

Ferric sulfate has certain advantages as a coagulant. The floc produced is denser than alum floc and is precipitated over a wide pH range (3.5 up). It does not form a soluble salt at high pH as does alum. On the other hand, ferric sulfate has certain disadvantages; it stains equipment; it presents problems in dissolving; its solution is corrosive; and it may be reduced to the soluble form (ferrous) in the presence of organic matter or reducing agents. The choice of a ferric sulfate chemical as a coagulant must be weighed against these factors.

Ferric sulfate reacts with the natural alkalinity of the water or with the added alkaline materials such as lime or soda ash. The reactions may be written to show precipitation of ferric hydroxide, although in practice, as with alum, the reactions are more complicated than this. The reactions are shown in Table 5-4.

Table 5-4

REACTIONS OF FERRIC SULFATE

$$Fe_2(SO_4)_3 + 3\ Ca(HCO_3)_2 \rightarrow 2\ Fe(OH)_3 \downarrow + 3\ CaSO_4 + 6\ CO_2 \uparrow$$

$$Fe_2(SO_4)_3 + 6\ NaHCO_3 \rightarrow 2\ Fe(OH)_3 \downarrow + 3\ Na_2SO_4 + 6\ CO_2 \uparrow$$

$$Fe_2(SO_4)_3 + 3\ Na_2CO_3 + 3\ H_2O \rightarrow 2\ Fe(OH)_3 \downarrow + 3\ Na_2SO_4 + 3\ CO_2 \uparrow$$

$$Fe_2(SO_4)_3 + 6\ NaOH \rightarrow 2\ FE(OH)_3 \downarrow + 3\ Na_2SO_4$$

$$Fe_2(SO_4)_3 + 3\ Ca(OH)_2 \rightarrow 2\ Fe(OH)_3 \downarrow + 3\ CaSO_4$$

In tests with pure solutions, optimum phosphate precipitation occurred in the 4.5-5 pH range (2); however, in wastewater good phosphate removals occur at least up to pH 8 with ferric salts.

5.2.5 Ferric Chloride

Ferric chloride ($FeCl_3$) is used in wastewater treatment as a coagulant, phosphate precipitant and as a sludge conditioner. It is available commercially in the anhydrous, crystal hydrated and liquid forms (6). Table 5-5 shows characteristics of the three forms of ferric chloride.

Table 5-5

CHARACTERISTICS OF FERRIC CHLORIDE

	Anhydrous	Crystal	Liquid
Composition	$FeCl_3$	$FeCl_3 \cdot 6H_2O$	30-46° Bé
$FeCl_3$ Content	96-97%	60%	35-45%
Fe Content	33-33.3%	20.5%	12-15.4%
Form	Granular	Lump	Solution
Color	Green-black	Yellow	Yellow-brown
Weight per cu ft	85-90 lb	60-64 lb	11.2-12.4 lb per gal
Hygroscopic	Very	Very	
Melting Point	577°F	98.6-102.2°F	14°F±
Heat of Soln & Hyd. Btu per lb	353	38.2	–

All forms of ferric chloride are hygroscopic and will absorb enough moisture from the air to spontaneously form solutions. The solutions are acidic and, therefore, highly corrosive.

Anhydrous ferric chloride is shipped in 150 and 350 lb non-returnable steel drums. Once drums have been opened, they should be completely emptied or tightly resealed to prevent absorption of moisture and liquefaction. Crystal ferric chloride is shipped in 100 lb kegs or 400 or 450 lb drums. Storage should be in a cool, dry place at less than 100°F to prevent melting of the solids. Once opened, containers of crystal ferric chloride should be completely emptied. Liquid ferric chloride solution is shipped in rubber lined tank cars or trucks. Because of freight costs, this mode of transportation is economical only if the point of use is near the point of manufacture.

a) Handling and Feeding

Ferric chloride must be fed as a solution because of its hygroscopic nature. Positive displacement liquid feeders should be used. Orifice-type solution feeders are not recommended due to the varying viscosity and specific gravity of ferric chloride solutions with changes in temperature. Solution characteristics are shown in Table 5-6.

Table 5-6

SOLUTION CHARACTERISTICS

	Specific Gravity				Solution Table at 20°C (68°F)	
% $FeCl_3$	15°C 59°F	20°C 68°F	25°C 77°F	30°C 86°F	lb per gal water	lb per gal solution
10	1.0867	1.0851	1.0853	1.0817	0.927	0.905
20	1.1834	1.1820	1.1805	1.1786	2.06	1.973
30	1.2935	1.2910	1.2885	1.2850	3.73	3.232
35	1.3555	1.3530	1.3505	1.3475	4.50	3.952
40	1.4205	1.4175	1.4145	1.4115	5.55	4.733
45	1.4885	1.4850			6.75	5.577

NOTES:
To convert to other temperatures, multiply by (Sp.G at T/Sp.G at 20°C).

To convert Solution Table for use with $FeCl_3 \cdot 6H_2O$, (crystals), multiply by 1.666. The rate of solution is slower for the crystal form, but any desired concentration of $FeCl_3$ may be obtained.

b) Dissolvers and Dissolving

Dissolving or solution tanks for either anhydrous or crystal ferric chloride should be sized to hold a 24-hour supply in small plants. In plants where there are three shifts, a supply of 8 to 12 hours should be provided so that the task of dissolving chemical need not be done more than once a shift. These tanks should be designed so that an entire drum of the size used in the plant can be emptied at one time. A rack or grid should be provided to prevent the chemical from piling up in the bottom of the tank. Agitation must be provided to give thorough distribution of the chemical in the solution.

Anhydrous ferric chloride has a high heat of solution and the temperature of the solution will rise as the chemical is dissolved. This temperature rise is 3.75°F for each one percent solution. Thus, a 40 percent solution (the maximum recommended) will show a temperature rise of 150°F. This fact is important in the choice of materials for lining tanks and in safe handling procedures. Crystal ferric chloride ($FeCl_3 \cdot 6\ H_2O$) has a much lower heat of solution, which presents no problem.

Hydrometer readings of ferric chloride solutions are not dependable indications of the concentration (%$FeCl_3$) because temperature and impurities are factors. Determine solution strengths by chemical analysis or by adding known weights of chemical to known volumes of water as indicated in Table 5-6. If desired, concentrated solutions may be diluted before or during feeding.

One satisfactory method of putting the anhydrous material into solution from the small drums is as follows: Roll the drum over the solution tank, which has its top flush with or slightly below the floor level; punch a hole in the top and another in the bottom of the drum; insert a hose into the top hole and allow water to trickle through, and ferric chloride solution will trickle out of the bottom hole. When the drum is empty, add water to solution tank to make up desired concentration. The tank may be of concrete; it should be provided with an agitator or stirring paddle. No grid is necessary in the tank, but a suitable support must be provided for the drum over the tank.

Rubber or rubber-lined vessels and equipment are most common. Stoneware, glass, Bakelite and other resinous plastics also may be used. Among the metal alloys, Hastelloy "C" or Durichlor resist all concentrations at room temperatures. Rubber linings should be protected against abrasion and high temperatures. Wooden grids and wooden tanks are slowly attacked by a shrinking action of ferric chloride; grids may require replacement and tanks may require tightening of the staves to prevent leakage. Concrete is frequently used without coating, but is slowly attacked unless coated with plastic, asphaltic paint, rubber or acid-proof brick.

If handled carelessly, anhydrous ferric chloride may produce dust which will irritate eyes and throats of workmen. It may spatter when put into water thus causing undue hazard. In concentrated form or solution, ferric chloride may be injurious, and all splashes on clothing and skin should be washed off immediately with water. If splashed in the eyes, they should be flushed with water, followed by a solution of boric acid and attention from a physician.

All forms of ferric chloride stain badly. Floors, walls, etc., should be protected by proper paints. Workmen should wear rubber gloves, rubber aprons and goggles.

5.2.6 Ferrous Coagulants

Ferrous salts are effective as coagulants for solids removal and phosphate precipitation. Granular ferrous sulfate may be used, however, the cheapest source of Fe^{+}_{2} in many localities is waste pickle liquor from the steel industry. Ferrous sulfate produced from pickling with sulfuric acid has been the most common form but ferrous chloride from hydrochloric acid pickling is also available. The pickle liquor is quite variable in composition depending on operation of the pickling process. Iron content of the ferrous sulfate liquors ranges up to about 22% $FeS0_4$ and free acid ranges from 0.5% to 15%. A typical ferrous chloride liquor used at Mentor, Ohio, contained 7 to 11% Fe and 0.4 to 0.6% free acid.

Special handling and feed procedures may be required with pickle liquor. Some ferrous sulfate liquors must be stored in heated tanks or be diluted to prevent crystals of $FeS0_4$ from forming and blocking lines and feeders. Information for designing facilities for pickle liquor should be obtained from the supplier.

Ferrous salts may be added to raw wastewater to improve removal of solids and phosphorus in the primary, or they may be added to the aeration tanks of activated sludge plants. When used in primary treatment it is generally necessary to add a source of alkalinity as a conditioning agent to produce a settleable floc. Either lime or sodium hydroxide are suitable for this purpose. A polyelectrolyte is also frequently necessary in this type of application. The dosage of ferrous salts should also be controlled to avoid the appearance of excessive iron residual in the effluent. If added to activated sludge, the applied oxygen oxidizes the excess Fe^{+2} to Fe^{+3} which precipitates excess iron as the insoluble $Fe(OH)_3$. Plant scale evaluations of ferrous salts have been run at Grayling and Lake Odessa, Michigan (7), Mentor, Ohio (8), Texas City, Texas (9), and Milwaukee, Wisconsin (10).

5.2.7 Lime

Hydrated lime [$Ca(OH)_2$] is used in wastewater treatment for coagulation of solids, precipitation of phosphate, pH adjustment and sludge conditioning (11). Dolomitic lime, which contains a substantial content of magnesium, may also be used for some purposes. Lime may also be purchased as anhydrous quicklime, but is slaked before use. The slaking process involves the addition of sufficient water in special equipment to hydrate the CaO or quicklime.

Table 5-7

CHARACTERISTICS OF LIME

	Hydrated Lime
Composition	$Ca(OH)_2$
$Ca(OH)_2$	82-99%
CaO Content	62-78% (Std. 66%)
$Mg(OH)_2$ Content	0
Form	Powder (200-400 mesh)
Color	White (tan)
lb/cu ft	30 to 50
Hygroscopic	Slightly
Solubility - gm/100 ml	0.18 @ 0°C 0.16 @ 20°C 0.15 @ 30°C
pH of Sat. Soln.	12.4
Angle of Repose	Varies

NOTE: Angle of repose depends on packing pressure to which chemical is subjected, and varies from 90° to zero, as aerated hydrated lime will flow like water.

Lime can be purchased in bulk in both car and truckload lots. It is also shipped in 50 and 100 lb multiwall moisture-proof paper bags. Storage of bagged lime should be indoors, in a dry place, to avoid absorption of moisture. Bulk lime is stored dry, inside or out, in air-tight concrete or steel bins. To ensure emptying of the bins, agitation should be provided to break arch formation. Bagged quicklime should not be stored more than 60 days due to moisture absorption through the multiwalled bags and the resultant caking of the lime.

Bulk quicklime may be stored indefinitely if the bins are air-tight. Hydrated lime may be stored for several months in a dry location.

Quicklime should be fed gravimetrically to slakers constructed of mild steel. Slaking time should be about 3 to 5 minutes with the addition of slaking water in a weight ratio of 2:1 to 6:1. Generally, dissolvers for dry lime feeders are designed to provide 3 to 5 minutes detention when forming a 6% slurry at the maximum feed rate required. Lime slurry is easier to transport, gives better dispersion in water, and by wetting all the particles assures that none will settle out in the treatment basin.

Lime slurry pumps should be of the open impeller centrifugal type with a cast iron body and impeller, with bronze trim. Tanks should be mild steel with a mild steel propeller-type mixer. Rubber hose slurry lines should be supported in pipe channels or by angle irons. Cast iron or mild steel pipe may be used. Valves should be cast iron or mild steel. Lime can be transported to various locations in the plant by conventional dry powder conveying systems. An efficient dust collecting system is strongly recommended at lime handling points. A dry pickup vacuum cleaner should be employed for removing dust around and on unloading equipment and slakers. This vacuum cleaner should be emptied promptly after each use.

When quicklime and water are mixed, enough heat is generated to endanger nearby flammable materials. Lime or quicklime is a strong caustic irritant affecting eyes, mucous membranes, and upper respiratory tract. When handling dry quicklime, proper clothing should be worn. Recommended are those made of denim with long sleeves and a hat or bandana. Trouser cuffs and shirt sleeves should be tied. Exposed skin areas should be covered with a protective cream. Goggles or a face shield should be worn. A suitable dust mask should be worn when handling lime as prolonged exposure to dust can cause lung damage. Solutions of hot lime suspension can cause severe burns and eye injuries. Workmen should take extreme precautions and always wear goggles or face shields when inspecting and cleaning lime slakers.

Lime is somewhat different from the hydrolyzing coagulants. When added to wastewater, it increases pH and reacts with the carbonate alkalinity to precipitate calcium carbonate. It also reacts with orthophosphate to precipitate gelatinous calcium hydroxyapatite. If sufficient lime is added, magnesium hydroxide will be precipitated to act as an effective coagulant for solids. These reactions are as follows:

$$Ca^{+2} + HCO_3^- + OH \longrightarrow CaCO_3 \downarrow + H_2O$$

$$5Ca^{+2} + 4\ OH^- + 3\ H\ PO_4^{-2} \longrightarrow Ca_5\ (OH)\ (PO_4)_3 \downarrow + 3\ H_2O$$

$$Mg^{+2} + 2\ OH^- \longrightarrow Mg(OH)_2 \downarrow$$

The required lime dosage to reach a given pH is a function of the alkalinity of the wastewater, as shown in Figure 5-1 (12).

In EPA's pilot plant at the District of Columbia's Blue Plains Plant (13, 14, 15, 16), the lime precipitation system gives excellent removals of solids and phosphorus. It may employ either a two-stage high lime process with lime addition to pH 11.5 in the first stage followed by recarbonation to pH 9.5-10 with flocculant additives in the second stage, or a single stage high lime process at pH 11.5 with sodium carbonate added to water to reduce the excess calcium ions. Poorer phosphorus removals of 77% occurred in the winter with single stage operation. The two-stage process with flocculants maintained 90% removals of phosphorus under winter conditions. With effective clarification followed by dual-media filtration at 3 gal/min/sq ft, residual turbidities of approximately 0.5 units were maintained with filter runs averaging 59 hours. Other locations where lime is being used for treatment of wastewater are at South Lake Tahoe, California (17),and in South Africa (18).

Lime sludge may be thickened, dewatered and calcined to convert the calcium carbonate to lime which is suitable for reuse. This is economical only in plants with flows greater than about 10 MGD. The cost of recovering lime is comparable in cost to purchasing new lime, but there may be a considerable savings in sludge disposal costs. At the present time, lime is the only coagulant which can be economically recovered for reuse.

5.2.8 Polymeric Flocculants

Addition of the hydrolyzing metal coagulants to wastewater often results in a small slow-settling floc or precipitate of phosphorus. Additional treatment is required to produce a water with low residual suspended solids. Polymeric flocculants have proved to be quite beneficial in aggregating the particles to a settleable size and increasing the shear strength of the floc against hydraulic breakup.

Polymeric flocculants are high molecular weight organic chains with ionic or other functional groups incorporated at intervals along the chains. Because these compounds have characteristics of both polymers and electrolytes, they are frequently called polyelectrolytes. They may be of natural or synthetic origin.

All synthetic polyelectrolytes can be classified on the basis of the type of charge on the polymer chain. Thus polymers possessing negative charges are called anionic while those carrying positive charges are cationic. Certain compounds carry no electrical charge and are called nonionic polyelectrolytes. Because of the great variety of monomers available as starting material and the additional variety that can be obtained by varying the molecular weight, charge density and ionizable groups, it is not surprising that a great assortment of polyelectrolytes are available to the wastewater plant operator. A partial list is shown in Table 5-8 (20). Use of any specific polymer as a coagulant aid is of necessity determined by the size, density and ionic charge of the colloids to be coagulated. As other factors need to be considered, i.e. coagulants used, pH of the system, techniques and equipment for dissolution of the polyelectrolyte, etc., it is mandatory that extensive jar testing be performed to determine the specific polymer that will perform its function most efficiently.

It is now generally agreed that a bridging mechanism accounts for the flocculation behavior of these compounds. In its simplest form the theory postulates that the polymer molecules attach themselves to the surfaces of the suspended particles at one or more sites and that

Figure 5-1

LIME REQUIREMENT FOR pH $\geq$ 11.0 AS A FUNCTION OF THE WASTEWATER ALKALINITY

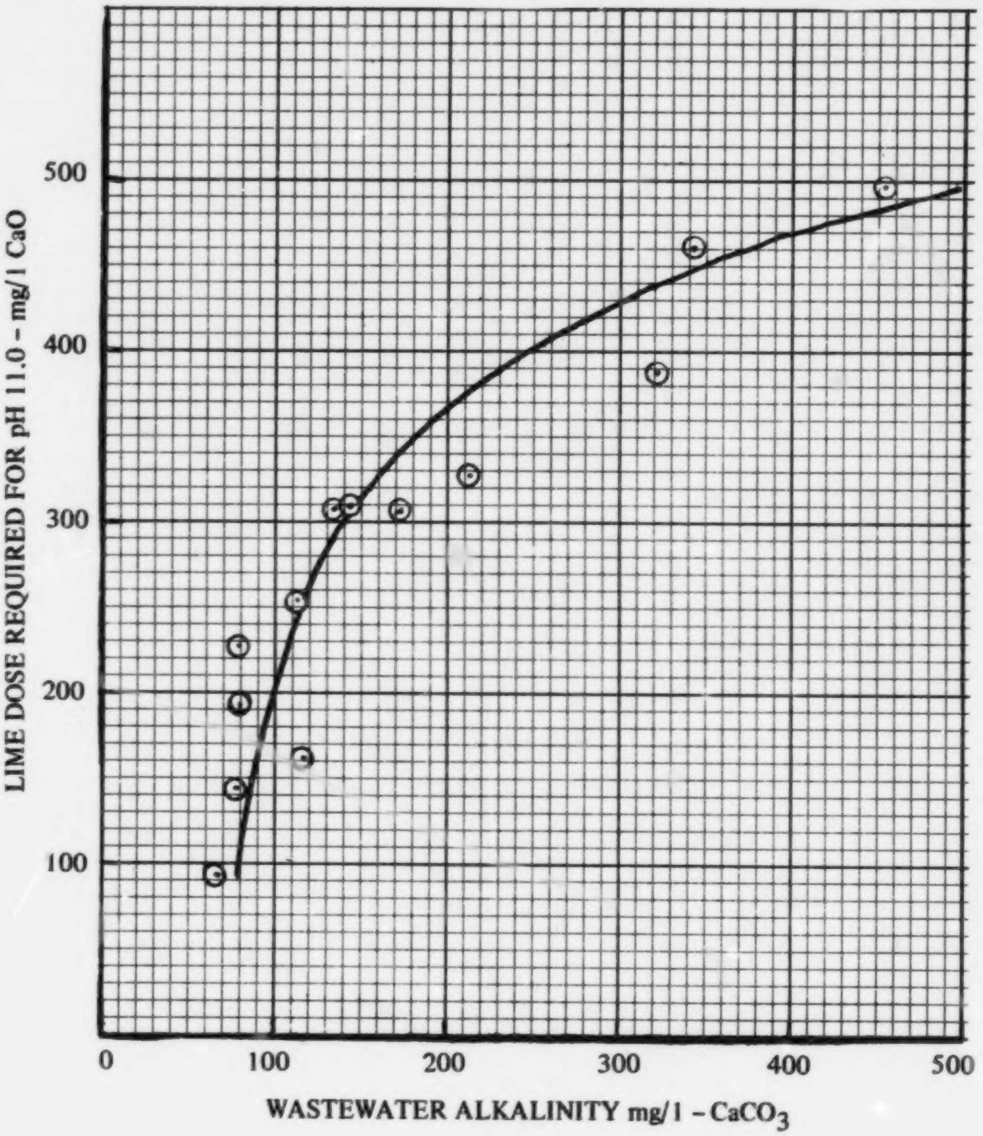


Table 5-8 DRY POLYELECTROLYTES

Polyelectrolyte	Type table 3	Bulk Density lb/cu ft			Flow table 4	Time to disperse into a coll. solution hour(s)	Solution—room temp.				Percent max solution concentration recommended
		Loose	Pack	Work			percent	Visc. cp	Sp gr	pH	
Aquafloc 409*	AP	25	34	28	CNKL	1-2	1	2350	1	4-5	1
Aquafloc 411*	AP	42	53	45	CNKL	1-2	2	480	1	4-5	2
Aquafloc 414	NP	48	59	61	CNKL	1-2	2	660	1	7	2
Aquafloc 418	CP				DPKL	1-2	2	23	1	2.4	2
Aquarid 49-702	CM	33	44	36	E S	1-2				7-9	
Calgon C-2256	CP	30	43	34	DLKP	½	1	75	1	6.4	1.5
Calgon C-2260	CP	24	35	28	DLKP	½	1	40	1	6.7	1.5
Calgon C-2270	CP			25	DLP	½	1	35	1	7.3	1.5
Calgon C-2300	NP	10	16	13	BLN	¾-1	0.5	24	1	7	0.5
Calgon C-2325	AP	10	16	13	BLN	¾-1	0.5	275	1	7	0.5
Calgon C-2350	AP	11	19	14	BLN	¾-1	0.25	425	1	7	0.25
Calgon C-2400	AP	10	16	13	ALKM	¾-1	0.25	740	1	7	0.25
Calgon C-2425	AP	16	28	22	BLKN	¾-1	0.25	800	1	7	0.25
Calgon WT-2600	CP	23	34	27	DKLP	½	1	38	1	4	1.5
Calgon WT-2630	CP	29	42	33	DKLP	½	1.5	20	1	4	1.5
Calgon WT2660 (ST-260)	CP	27	39	31	DKLP	½	0.5	20	1	4	0.5
Calgon WT-2690	NP	9	16	12	ALM	½	0.5	23	1	7	0.5
Calgon WT-2700	AP	10	18	14	DLP	½	0.25	80	1	7.5	0.25
Calgon WT-2900	AP	8	13	10	AKLN	½	0.25	160	1	7.5	0.25
Calgon WT-3000	AP	16	29	22	AKLN	½	0.25	250	1	7.5	0.25
Hamaco 196*	S	20	25	21	DLP	½-1	1	<50	1	<6-7	2
Hercofloc 810	CP	22	31	25	EKLR	1-2	0.5	400	1	<6-7	0.5
Hercofloc 812	CP	21	31	24	EKLR	1-2	0.5	250	1	<6-7	0.5
Hercofloc 818	AP	30	40	33	DKLN	1-2	0.5	1000	1	<8-9	0.5
Ionac NA-710	AP	38	47	40	CNL	½-1	0.2	150	1	5-6	1
Jaguar Plus	CG	31	40	22	FLPR	1-2	1	800	1	8-9	1
Magnifloc 530C	CP										0.5
Magnifloc 820A	AD	34	40	35	CLP	1-2	1	160	1	4.2	1
Magnifloc 835A	AD	30	42	34	DKP	½-1	0.1	450	1	6-7	0.1
Magnifloc 836A	AD	27	35	29	CKP	½-1	0.1	360	1	6-7	0.1
Magnifloc 837A	AD	28	36	30	DLP	½-1	0.5	620	1	5	0.5
Magnifloc 865A	AD	40	42	50	CLN	1½ 2	1	130	1	4.1	1
Magnifloc 870A	AD			28		⅛	0.5	38	1	7.5	1
Magnifloc 875A	AD			28		¼	0.5	38	1	7.5	1
Magnifloc 880A	AD			26		1-2	0.5	20	1	5.5	1
Magnifloc 900N	ND	32	40	33	CLN	1-2	1	150	1	4.7	1
Magnifloc 901N	ND					1-2	1	300	1	4.7	1
Magnifloc 902N	ND					1-2	1	200	1	4.7	1
Magnifloc 905N	ND	27	35	29	CLN	1-2	1	500	1	4.7	1
Nalcolyte 610	P	26	40	31	DKLP	½-1	0.5	700	1	5.1	1
Nalco 633-HD	CP	42	68	52	DJLN	¼-½	2	190	1	3	2
Nalco 636-HD	CP	45	53	47	DLN	¼-½	1.5	33	1		1.5
Nalco 635	AP	38	50	41	DLN	¾-1	0.25	2500	1	8.5	0.25
Nalco D-2339	AP	39	53	43	DJLS	½-1	0.3	2000	1	7.0	0.5
Nalcolyte 675	AP	35	50	40	DLN	½-¾	0.25	2500	1	8.5	0.25
Polymer F3	AG	32	40	34	DLN	1-2	4	800	1	9.5	4
Polyfloc 1100	AD	34	40	35	CKN	1-2	0.5	2000	1	8.5	0.5
Polyfloc 1110	AD	36	42	37	CKN	1-2	0.5	2400	1	7.5	0.5
Polyfloc 1120	AD	36	42	37	CKN	1-2	0.5	3500	1	8.8	0.5
Polyfloc 1130	AD	35	48	39	DKP	1-2	1	3000	1	8.8	0.5
Polyfloc 1150	AD	33	45	36	FJR	½-1	1	1000	1	6-7	1
Polyfloc 1160	CD	34	40	35	CJN	½-1	1	1300	1	5-6	1
Purifloc A-23	CD	42	53	45	BKP	1-2	0.1	950	1	10	0.5
Superfloc 128	ND	28	33	29	CLN	1-2	1	750	1	6-7	1
Tychem 8024*	ND			40		1-2	1.0	200	1	5.5-6	1.0
Tychem 8013	ND	33	43	36	CLN	1-2	1.0	750	1	6	1
Zeta Floc C*	BC[1]	48	68	54	EKR	¼-½				8.2	1
Zeta Floc O*	BN[1]	52	74	59	EJR	¼-½				8.2	1
Zeta Floc K (+KMnO₄)*	BC[1]	48	68	54	EKR	¼-½				8.5	1
Zeta Floc S	BC[1]	54	78	61	E S	¼-½					1
Zeta Floc WA*	BA[1]	54	78	61	E S	¼-½					1
Zeta Flux WN	BN[1]	54	78	61	E S	¼-½					1

*Approved by USPHS for potable water use.

[1] Aluminum Silicate added.

Table 5-8 (Continued) LIQUID POLYELECTROLYTES

Polyelectrolyte	Type table 3	Solution strength percent	Sp gr room temp. (about)	Viscosity—cp room temp.	pH (about)	Dilution
Aquafloc 403	AH	Full	1.12	100	9.6	4:1
Aquafloc 405	CH	Full	1.06	1,000	6.3	10:1
Aquafloc 407	NH	Full	1.00	10,000	4.3	10:1
Aquafloc 408*	AH	Full	1.01	1,500	3.4	10:1
Aquafloc 410	CH	Full	1.03	1,000	10.2	10:1
Aquafloc 412	CH[1]	Full	1.25	50	1.0	4:1
Aquafloc 415	AH	Full	1.03	1,000	11.5	10:1
Aquarid 49-700	CM	Full	1.1	100-500	7.8	10:1
Aquarid 49-701	CM	Full	1.1	25-150	7.8	10:1
Aquarid 49-703	AH[3]	Full	1.2	200-500	11.5	20:1
Cat-Floc (WT-2870)*	CH[2]	Full	1.025	2,000	4.2	<10:1
Magnifloc 521-C	CH	Full	1.15	225-325	4.5	<10:1
Nalcolyte 603	CH	Full	1.6	80	8	<10:1
Nalcolyte 607	CH	Full	1.17	50	7.5	<10:1
Natron 86, 18 percent	CH	Full	1.06	300-500	3	< 0.5
Polyfloc 1170	H	Full	1.06	50	9	
Polyfloc 1175	H	Full	1.08	700	9	

*Approved by USPHS for potable water use.

[1] Plus a primary coagulant.

[2] A linear homopolymer of diallyldimethyl ammonium chloride.

[3] Polyaromatic.

TYPES

A — Anionic
As — Slightly Anionic
B — Bentonite Clay or Clay, natural, colloidal-like type
b — plus Bentonite
C — Cationic
D — Polyacrylamide, Synthetic, High M.W., Polyelectrolyte Polymer
E — Polyacrylonitrile, Synthetic Polyelectrolyte
F — Sulfonated Polymer
G — Guar Gum, Polysaccharide, Natural Polymer
H — High M.W., Organic Polymer
J — Alkyl Guanidineamine Complex
K — Sodium Alginate or Algin Derivative, Natural Polymer
L — Leguminous Seed Derivative, Natural Polymer
M — Polyamine, Synthetic, High M.W., Polyelectrolyte Polymer
N — Nonionic
P — Synthetic High M.W., Polyelectrolyte Polymer
R — Polyacrylamide and Carboxylic Group
S — Starch, derivative, mdoified, etc., Natural Polymer
T — Synthetic Polymer and Caustic Soda
U — Sodium Carboxymethylcellulose, Natural Polymer
X — Ethylene Oxide Polymer
Y — Carboxyl Polymer
2 — Biocolloid + Inorganic Coagulant + Caustic Soda
3 — Hydrophylic Colloid + Pregelatinized Starch in Alkalai
4 — Aluminum Hydroxide + Complex Organic Polymer
5 — Alumina + Polymer + Caustic Soda
6 — Polyacrylic Acid or Polyacrylate of Sodium or Ammonium
7 — Aluminum Hydrate + Caustic Soda
8 — Alkalai Concentrate + Metallic Ions
9 — Chemically Modified Natural Polymer

FLOW

A. Soft flakes, may hang up if packed excessively in a confining area, otherwise free flowing. Usually will not need aid (vibration or agitation).
B. Powdered, soft flakes, hang up if packed excessively in a confining area, may or may not need aid according to rate of feed, etc.
C. Soft granules, sometimes fibrous or flattish, may hang up if packed excessively in a confining area, otherwise free flowing. Usually will not need aid.
D. Powdered, soft granules, sometimes fibrous or flattish, hang up if packed excessively, may or may not need aid, according to other factors.
E. Granular, fluid powder, will arch if packed and can be fluidized or is floodable (to very floodable). Needs aid and may need rotor, according to rate, etc.
F. Granules and powder, will arch and can be fluidized. Needs aid and could need rotor, etc.
G. Cohesive powder and granules, will arch, but will not flood. Needs aid.
H. Cake up at room relative humidity.
J. Tendency to cake (or mass) at higher relative humidity.
K. Cake at higher relative humidity.
L. Moisture absorption, may lessen flowability.
M. Practically no dust.
N. Very little dust.
P. Some dust.
R. Dusty.
S. Very dusty.

TRADE NAMES

Aquarid — Reichhold Chemicals
Colgon C — Colgon Corp.
Colgon WT — Colgon Corp.
Cat Floc — Colgon Corp.
Hamco — A. E. Staley Manufacturing Co.
Hercofloc — Hercules, Inc.
Ionac NA-710 — Ionac Chemical
Natron — National Starch and Chemical Corp.
Polyfloc — Betz Laboratories, Inc.
Polymer F3 — Stein Hall
Tychem — Standard Brands Chem. Ind., Inc.
Zeta Floc — Narvon Minnig and Chemical Co.

part of the long chain extends out into the bulk of the solution. The free end of the molecule is then able to adsorb onto another suspended particle when contact is made, thus forming a bridge or link between two particles of suspended solids. The progressive linking of more and more particles results in an ever-increasing size of floc whose eventual size is limited by its ability to withstand the hydraulic shear gradient imposed upon it by agitation or turbulence.

Anionic polymers have proved to be most effective when used with alum or iron coagulants The anionics have also been used as sole additives to improve removals of solids in primary treatment, but results have been marginal. Cationic polymers when used alone will frequently produce excellent clarification of raw wastewater, but generally require high dosages. The cationics find their greatest application in conditioning of sludges for dewatering.

Polymers are used in very small doses, usually less than 1 mg/1. The dosage range in which the polymers are effective is usually limited. An overdose, in addition to the increased cost of chemicals, will frequently restabilize the solids so that they cannot be settled out. A 10 MGD plant employing a 0.5 mg/1 dose of polymer would use 42 lbs of polymer per day. At typical polymer cost of $1.50/lb, the cost of polymer for the plant would be $63.00/day.

Polymers are usually shipped as dry powders and are converted to a liquid form at the plant site, although some are available as liquids. During storage the dry powders must not be allowed to pick up moisture. The dilute solutions of polymers are viscous and must be made up according to the manufacturer's directions. In general, the polymers are non-hazardous and require only the usual protection from dust when handling the dry material. The dosage of polymer may be accurately controlled by metering pumps.

5.3 Chemical Feeders

Chemical feed systems must be flexibly designed to provide for a high degree of reliability in light of the many contingencies which may affect their operation. Thorough waste characterization in terms of flow extremes and chemical requirements should precede the design of the chemical feed system. The design of the chemical feed system must take into account the form of each chemical desired for feeding, the particular physical and chemical characteristics of the chemical, maximum waste flows and the reliability of the feeding devices.

In suspended and colloidal solids removal from wastewaters the chemicals employed are generally in liquid or solid form. Those in solid form are generally converted to solution or slurry form prior to introduction to the wastewater stream; however, some chemicals are fed in a dry form. In any case, some type of solids feeder is usually required. This type of feeder has numerous different forms due to wide ranges in chemical characteristics, feed rates and degree of accuracy required. Liquid feeding is somewhat more restrictive, depending mainly on liquid volume and viscosity.

The capacity of a chemical feed system is an important consideration in both storage and feeding. Storage capacity design must take into account the advantage of quantity purchase versus the disadvantage of construction cost and chemical deterioration with time (19). Potential delivery delays and chemical use rates are necessary factors in the total picture. Storage tanks or bins for solid chemicals must be designed with proper consideration of the angle of repose of the chemical and its necessary environmental requirements, such as temperature and humidity. Size and slope of feeding lines are important along with their materials of construction with respect to the corrosiveness of the chemicals.

Chemical feeders must accommodate the minimum and maximum feeding rates required. Baker (19) indicates that manually controlled feeders have a common range of 20:1, but this range can be increased to about 100:1 with dual control systems. Chemical feeder control can be manual, automatically proportioned to flow, dependent on some form of process feedback or a combination of any two of these. The cost of the more sophisticated control systems is relatively low if proper sensors are available. If manual control systems are specified with the possibility of future automation, the feeders selected should be amenable to this conversion with a minimum of expense. An example would be feeders with external motors which could easily be replaced with a variable speed motor or drive when automation is installed (19). Standby or backup units should be included for each type of feeder used. Reliability calculations will be necessary in larger plants with a greater multiplicity of these units. Points of chemical addition and piping to them should be capable of handling all possible changes in dosing patterns in order to have proper flexibility of operation. Designed flexibility in hoppers, tanks, chemical feeders and solution lines is the key to maximum benefits at least cost (19).

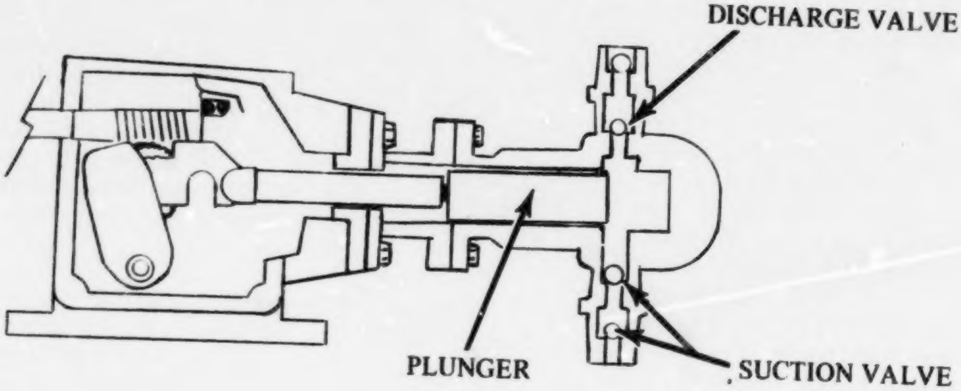
Liquid feeders are generally in the form of metering pumps or orifices. Usually these metering pumps are of the positive-displacement variety, plunger or diaphragm type. The choice of liquid feeder is highly dependent on the viscosity, corrosivity, solubility, suction and discharge heads, and internal pressure-relief requirements (20). Some examples are shown in the Figure 5-2. In some cases control valves and rotameters may be all that is required. In other cases, such as lime slurry feeding, centrifugal pumps with open impellers are used with appropriate controls (23). More complete descriptions of liquid feeder requirements can be found in the literature and elsewhere (20).

Solids characteristics vary to a great degree and the choice of feeder must be considered carefully, particularly in the smaller-sized facility where a single feeder may be used for more than one chemical. Generally, provisions should be made to keep all chemicals cool and dry. Dryness is very important, as hygroscopic (water absorbing) chemicals may become lumpy, viscous or even rock hard; other chemicals with less affinity for water may become sticky from moisture on the particulate surfaces, causing increased arching in hoppers. In either case, moisture will affect the density of the chemical and may result in under-feed. Dust removal equipment should be used at shoveling locations, bucket elevators, hoppers and feeders for neatness, corrosion prevention and safety reasons. Collected chemical dust may often be used.

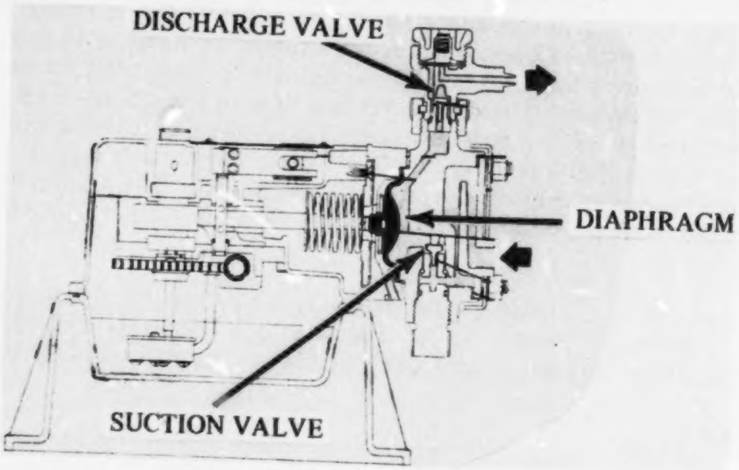
The simplest method for feeding solid chemicals is by hand. Chemicals may be preweighed or simply shoveled or poured by the bagful into a dissolving tank. This method is of economic necessity limited to very small operations, or to chemicals used in very weak solutions.

Because of the many factors, such as moisture content, different grades and compressibility, which can affect chemical density (weight to volume ratio), volumetric feeding of solids is normally restricted to smaller plants, specific types of chemicals which are reliably constant in composition and low rates of feed. Within these restrictions, several volumetric types are available. Accuracy of feed is usually limited to less than 97%.

One type of volumetric dry feeder uses a continuous belt of specific width moving from under the hopper to the dissolving tank. A mechanical gate mechanism regulates the depth of material on the belt, and the rate of feed is governed by the speed of the belt and/or the height of the gate opening. The hopper normally is equipped with a vibratory mechanism to reduce arching. This type of feeder is not suited for easily fluidized materials.

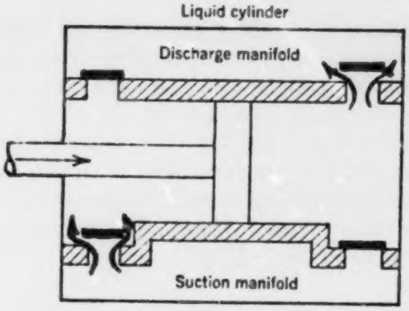

DISCHARGE VALVE
PLUNGER
SUCTION VALVE

PLUNGER PUMP (Courtesy of Wallace & Tiernan)

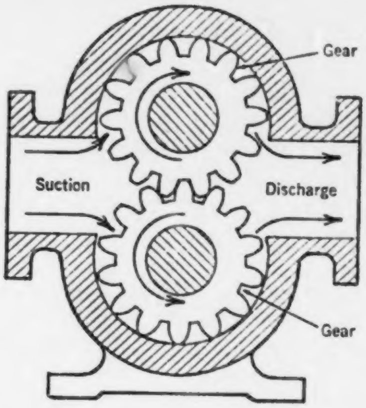

DISCHARGE VALVE
DIAPHRAGM
SUCTION VALVE

DIAPHRAGM PUMP (Courtesy of Wallace & Tiernan)

Figure 5-2 POSITIVE DISPLACEMENT PUMPS


Liquid cylinder
Discharge manifold
Suction manifold

PISTON PUMP (21)


Gear
Suction
Discharge
Gear

GEAR PUMP (21)

Figure 5-2 (Continued)

Another type employs a screw or helix from the bottom of the hopper through a tube opening slightly larger than the diameter of the screw or helix. Rate of feed is governed by the speed of screw or helix rotation. Some screw-type designs are self-cleaning, while others are subject to clogging. Figure 5-3 shows a typical screw-feeder.

Most remaining types of volumetric feeders generally fall into the positive-displacement category. All designs incorporate some form of moving cavity of a specific or variable size. In operation, the chemical falls by gravity into the cavity and is more or less fully enclosed and separated from the hopper's feed. The size of the cavity, and the rate at which the cavity moves and is discharged, governs the amount of material fed. The positive control of the chemical may place a low limit on rates of feed. One unique design is the progressive cavity metering pump, a non-reciprocating type. Positive-displacement feeders often utilize air injection to enhance flowability of the material. Some examples of positive-displacement units are illustrated in Figures 5-4 and 5-5 (22).

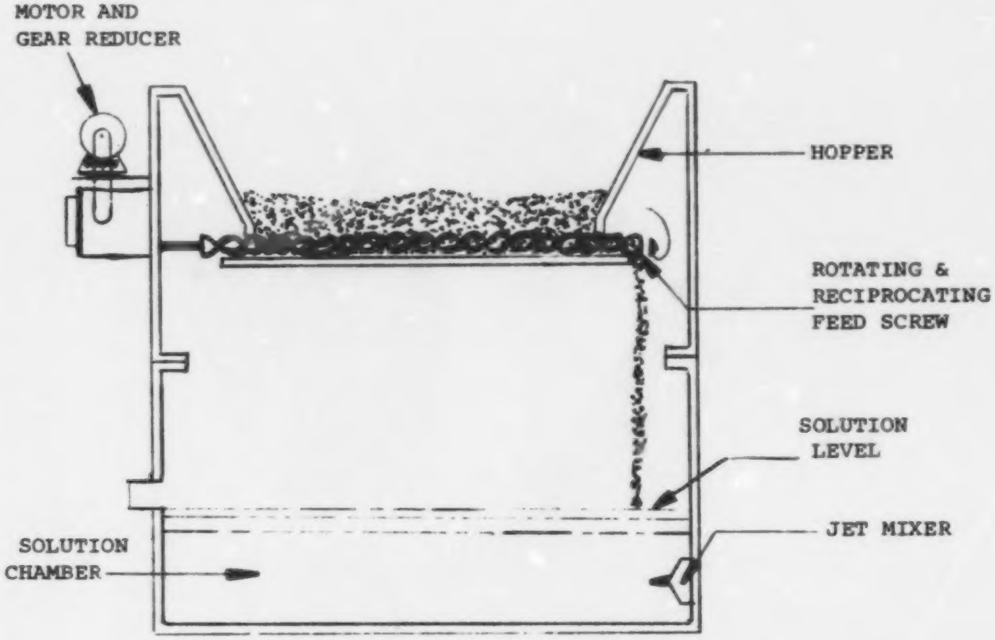
The basic drawback of volumetric feeder design, i.e., its inability to compensate for changes in the density of materials, is overcome by modifying the volumetric design to include a gravimetric or loss-in-weight controller. This modification allows for weighing of the material as it is fed. The beam balance types measures the actual mass of material. This is considerably more accurate, particularly over a long period of time, than the less common spring-loaded gravimetric designs. Gravimetric feeders are used where feed accuracy of about 99% is required for economy, as in large scale operations and for materials which are used in small, precise quantities. It should be noted, however, that even gravimetric feeders cannot compensate for weight added to the chemical by excess moisture. Many volumetric feeders may be converted to loss-in-weight function by placing the entire feeder on a platform scale which is tared to neutralize the weight of the feeder.

Good housekeeping and need for accurate feed rates dictate that the gravimetric feeder be shut down and thoroughly cleaned on a regular basis. Although many of these feeders have automatic or semi-automatic devices which compensate to some degree for accumulated solids on the weighing mechanism, accuracy is affected, particularly on humid days when hygroscopic materials are fed. In some cases, built-up chemicals can actually jam the equipment.

No discussion of feeders is complete without at least passing reference to dissolvers, as any metered material must be mixed with water to provide a chemical solution of desired strength. Most feeders, regardless of type, discharge their material to a small dissolving tank which is equipped with a nozzle system and/or mechanical agitator depending on the solubility of the chemical being fed. Insoluble materials, such as polyelectrolytes, may be carefully spread into a vortex spray or washdown jet of water immediately before entering the dissolver. Because this type of material does not truly dissolve, it is essential that the surface of each particle become thoroughly wetted before entering the feed tank to ensure accurate dispersal and to avoid clumping, settling or floating.

A dissolver for a dry chemical feeder is unlike a chemical feeding mechanism, which by simple adjustment and change of speed can vary its output tenfold. The dissolver must be designed for the job to be done. A dissolver suitable for a rate of 10 lb/hr may not be suitable for dissolving at a rate of 100 lb/hr. As a general rule, dissolvers may be oversized, but dissolvers for commercial ferric sulfate or lime slakers do not perform well if greatly oversized. An example of a typical unit of this type can be seen in Figure 5-6.

Figure 5-3 SCREW FEEDER


MOTOR AND
GEAR REDUCER
HOPPER
ROTATING &
RECIPROCATING
FEED SCREW
SOLUTION
LEVEL
JET MIXER
SOLUTION
CHAMBER

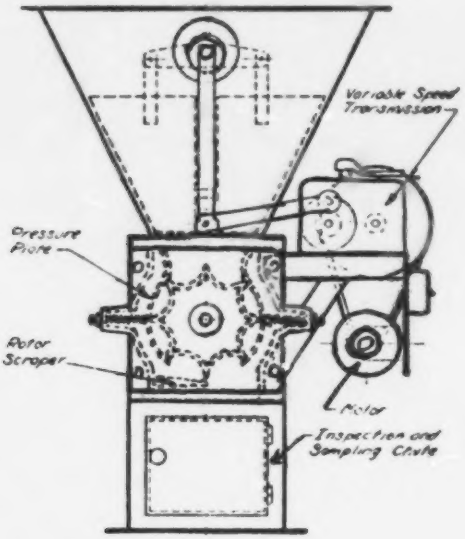

Variable Speed
Transmission
Pressure
Plate
Rotor
Scraper
Motor
Inspection and
Sampling Chute

Figure 5-4 POSITIVE DISPLACEMENT SOLID FEEDER – ROTARY (22)

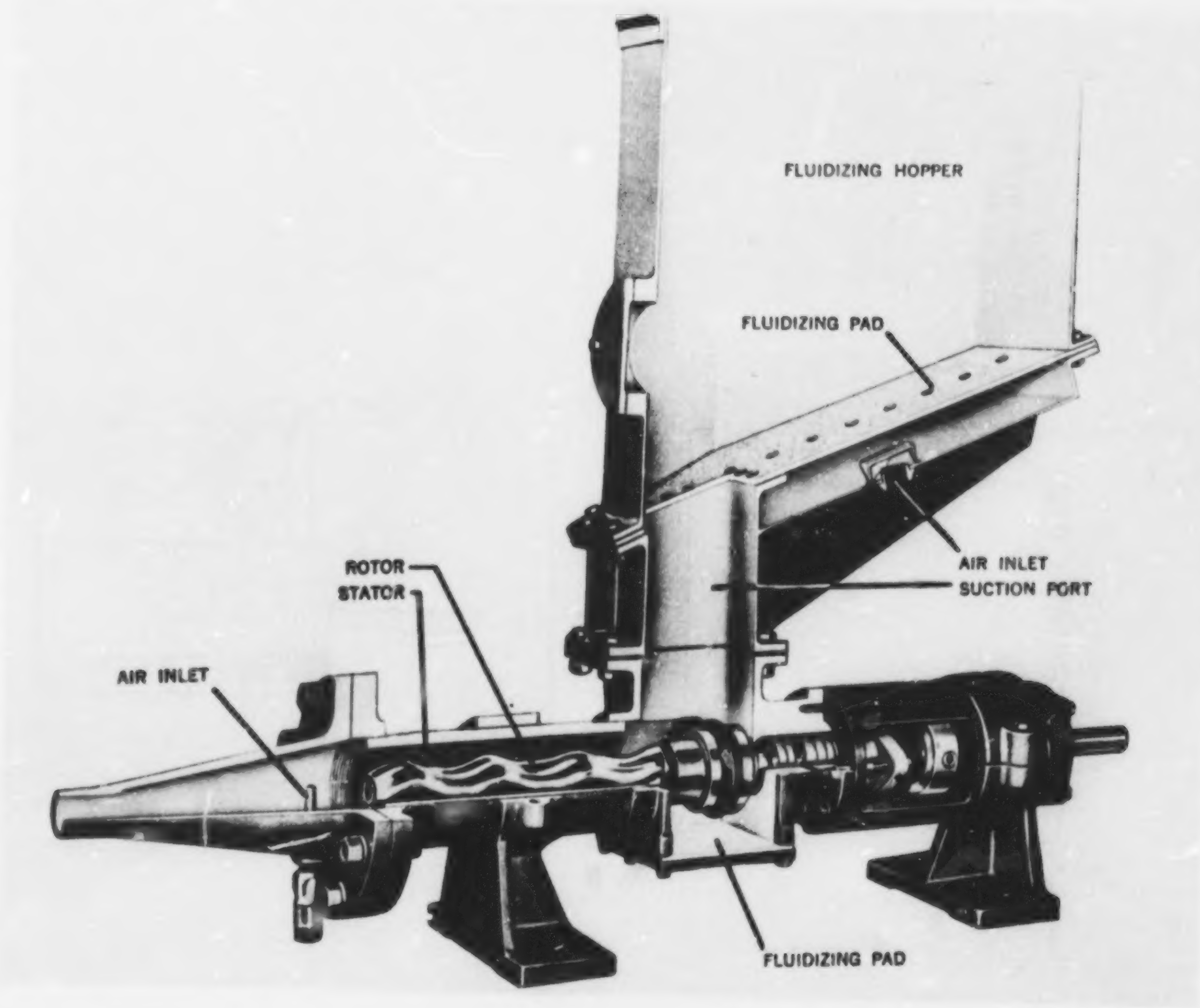

FLUIDIZING HOPPER
FLUIDIZING PAD
AIR INLET
SUCTION PORT
ROTOR
STATOR
AIR INLET
FLUIDIZING PAD

Figure 5-5 POSITIVE DISPLACEMENT POWDER PUMP
(Courtesy of Robbins & Meyers)

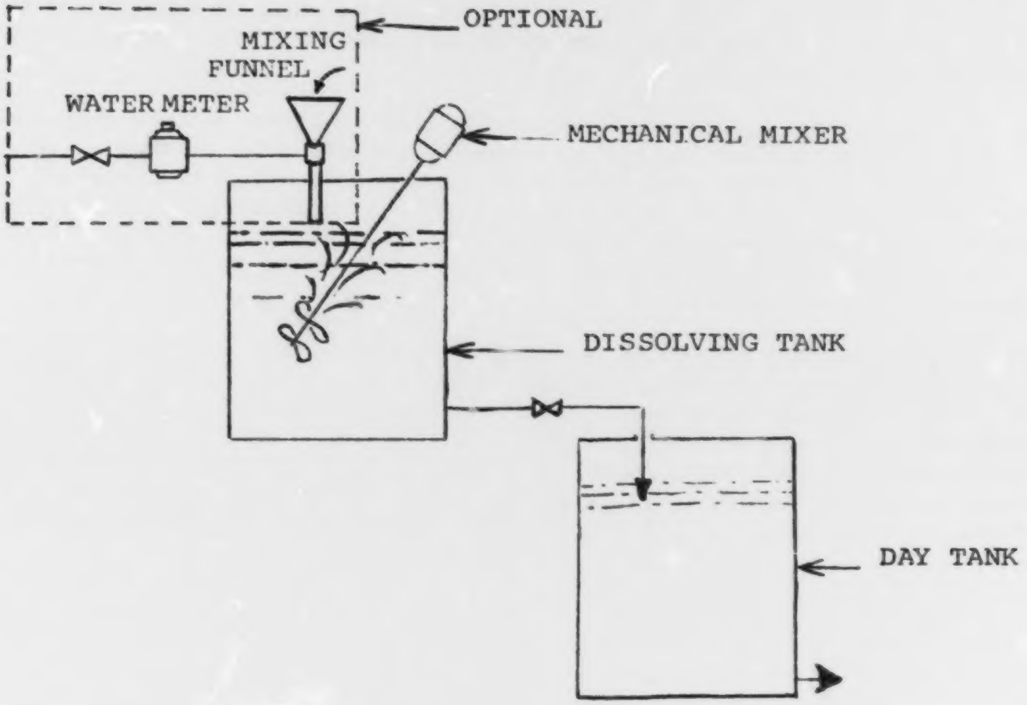

OPTIONAL
MIXING FUNNEL
WATER METER
MECHANICAL MIXER
DISSOLVING TANK
DAY TANK

Figure 5-6 DRY CHEMICAL DISSOLVER

It is essential that specifications for dry chemical feeders include specifications on dissolver capacity. A number of factors need to be considered in designing dissolvers of proper capacity. These include detention times and water requirements, as well as other factors specific to individual chemicals.

The capacity of a dissolver is based on detention time, which is directly related to the wettability or rate of solution of the chemical. Therefore, the dissolver must be large enough to provide the necessary detention for both the chemical and the water at the maximum rate of feed. At lower rates of feed, the strength of solution or suspension leaving the dissolver will be less, but the detention time will be approximately the same unless the water supply to the dissolver is reduced. When the water supply to any dissolver is controlled for the purpose of forming a constant strength solution, mixing within the dissolver must be accomplished by mechanical means, because sufficient power will not be available from the mixing jets at low rates of flow.

Specific factors as to recommended chemical feed rates per volume of water, detention times required, and necessary materials of construction are available in the literature (24), (25). Alum, lime and ferrous sulfate have been found to require about 5 minutes detention time at about 0.5 lb/gal. Ferric sulfate requires longer detention times (20 to 30 minutes) than the other granular chemicals. Hot water dissolvers are also available in order to decrease the required tankage. Further practical experience with a number of these chemicals is available in Culp and Culp (23).

The foregoing descriptions give some indication of the wide variety of materials which may be handled. Because of this variety, a modern facility may contain any number of a variety of feeders with combined or multiple materials capability. Ancillary equipment to the feeder also varies according to the material to be handled. Liquid feeders encompass a limited number of design principles which account for density and viscosity ranges. Solids feeders, relatively speaking, vary considerably due to the wide range of physical and chemical characteristics, feed rates and the degree of precision and repeatability required.

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CHAPTER 6

CHEMICAL PROCESSES

6.1 General

Chemical processes which are considered for suspended solids and phosphorus removal applications are usually of two types:

a. Traditional separate rapid-mixing, flocculation and sedimentation.

b. Solids-contact clarifiers.

The purpose of these chemical processes is to promote ideal conditions for the chemical reactions and for the separation of solids created as products of these reactions.

6.2 Chemical Mixing and Flocculation

Traditional water treatment systems have utilized rapid-mix and flocculation basins ahead of sedimentation tanks for chemical clarification. The basic equipment used in wastewater treatment is generally the same as that used in water treatment. The rapid mix is designed to provide a thorough and complete dispersal of chemical throughout the wastewater being treated to insure uniform exposure to pollutants which are to be removed. Detention times vary, but 10 to 30 seconds are usual design figures with velocity gradients of about 300 sec^{-1})(1). In-line blenders can be used as well as the traditional high-powered mixers which may require as much as 1 horsepower/MGD. In essence, the rapid mix performs two functions, the one previously noted (mixing) and a rapid coagulation. These functions are enhanced by increased turbulence (2), (3). Hudson and Wolfner (4) recommend that mixing equipment be of the variable-speed variety in order to promote optimum mixing conditions for the particular conditions of application. Some typical mixers are shown in Figures 6-1 and 6-2.

Flocculation promotes the contact, coalescence and size increase of coagulated particles (1). Flocculation devices vary in form, but are generally divided into two categories. These are mechanically-mixed and baffled flocculators. Typical units are illustrated in Figures 6-3 and 6-4. Baffled basins have the advantage of low operating and maintenance costs, but they are not normally used because of their space requirement, inability to be easily modified for changing conditions and high head losses. Most installations utilize horizontal or vertical shaft mechanical flocculators which are easily adjusted to changing requirements. These units are compared and discussed by Hudson and Wolfner (4) and by Camp (5). The location of these flocculators in series having progressively lower velocity gradients has been shown to produce large rapid-settling flocs (1), (4), (5).

Theories of flocculation are available in the literature (2), (3), (6). It appears from these studies that the criterion for proper flocculation lies in the dimensionless factor GCt, where G = velocity gradient (sec^{-1}), C = floc volume concentration (vol floc/vol suspension) and t = time of flocculation (sec). Values available in the literature for good flocculation in both solids-contact and conventional mixing-flocculation systems appear to fall in the range from 50 to 200 (6).

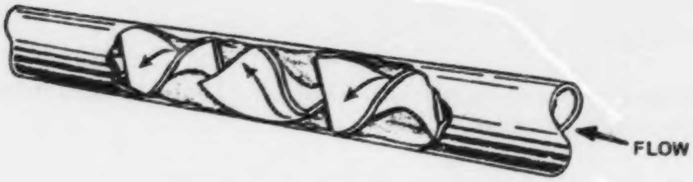

Figure 6-1 STATIC MIXER

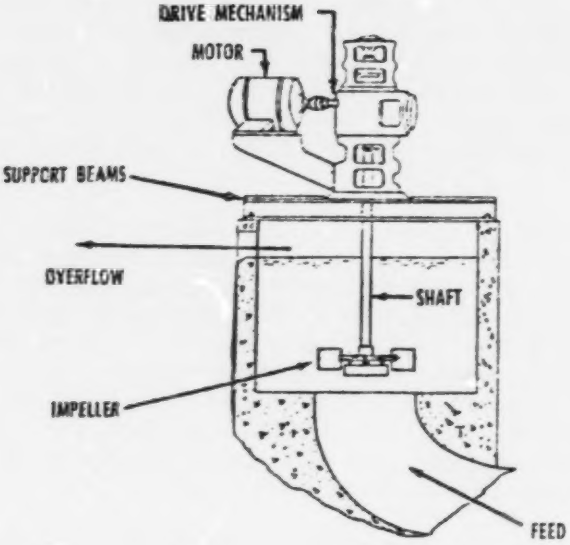
DRIVE MECHANISM
MOTOR
SUPPORT BEAMS
OVERFLOW
SHAFT
IMPELLER
FEED

Figure 6-2 IMPELLER MIXER

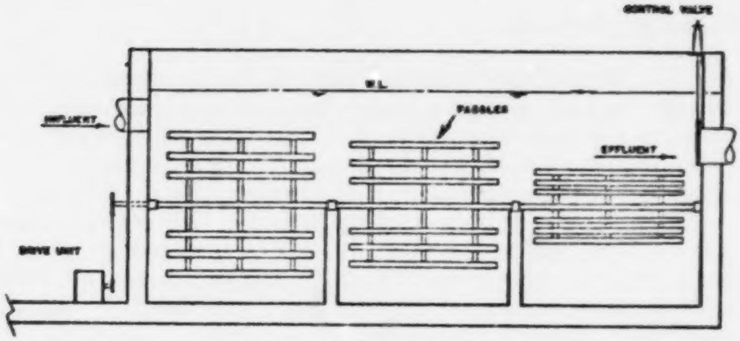
CONTROL VALVE
W.L.
INFLUENT
PADDLES
EFFLUENT
DRIVE UNIT

Figure 6-3 MECHANICAL FLOCCULATION BASIN

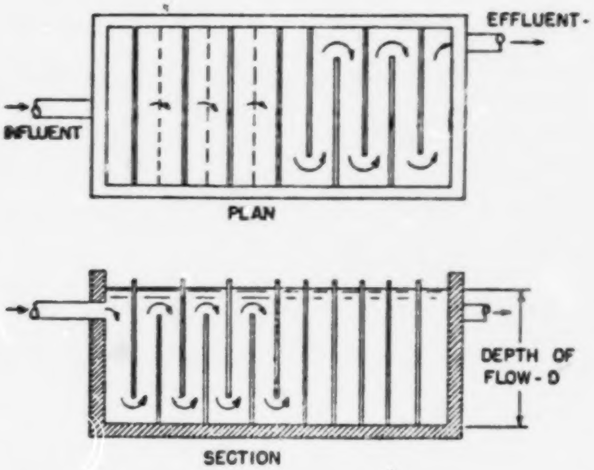
EFFLUENT
INFLUENT
PLAN
DEPTH OF FLOW - D
SECTION

Figure 6-4 BAFFLE FLOCCULATION BASIN

Culp and Culp (7) indicate their preference for the traditional rapid mix, flocculation, sedimentation approach owing mainly to its amenability to control of each unit process. At the South Tahoe plant, recycle of settled sludge to the head end of the system provides additional flexibility for enhanced flocculation by providing for possible increases in the floc volume concentration. Polymer addition is made after the wastewater has passed through the flocculation basin.

6.3 Solids-Contact Clarification

Solids-contact clarifiers have become popular for water and advanced wastewater treatment in recent years because of their inherent size reduction when compared to separate mixing, flocculation and sedimentation basins in series. Their use in water clarification and softening was carried over to waste treatment when chemical treatment of wastewaters was initiated. Theoretically, the advantage of reduced size accrues to their ability to maintain a high concentration of previously-formed chemical solids for enhanced orthokinetic flocculation or precipitation and their physical design, whereby three unit processes are combined in one unit. In practice this amounts to savings in equipment size and capital costs. The basin configurations and vertical flow patterns have been cited as conducive to reduced density currents and short-circuiting when compared to horizontal tanks (8).

Culp and Culp (7) have expressed disfavor with the sludge-blanket clarifiers for reasons which include possible anaerobic conditions in the slurry; lack of individual process control for the mixing, flocculation and sedimentation steps; and uncontrolled blanket upsets under varying hydraulic and organic loading conditions. The major allegation is the instability of the blanket, which has presented operational problems in the chemical treatment of wastewaters. Possibly the most effective method of control to date, other than close manual control, has been to minimize the blanket height to allow for upsets (9). The advantages of higher flow rates and solids-contacting are maintained, but the added advantage of the blanket is minimized. Another possibility which has not been fully evaluated is the use of sludge-blanket sensors for automatic control of solids wasting.

Tesarik (10) classified commercial sludge-blanket clarifiers into four categories:

a. Mechanically-agitated – central mixing with minimal fluidization of blanket due to low overflow rates.

b. Hydraulically-fluidized – central mixing with full fluidization due to high overflow rates.

c. Sludge circulation – slurry mixing with recirculation of settled solids to the slurry.

d. Unsteady discharge – pulsating feed of chemically-treated influent to a fluidized blanket.

In the United States, most of the systems used are of type c, which can be termed "solids-contact" clarifiers due to the intimate contacting of the influent stream with the recirculating slurry. The high concentration of solids in the recycled slurry provides for a rapid reaction. Typical solids-contact clarifiers are illustrated in Figures 6-5 and 6-6.

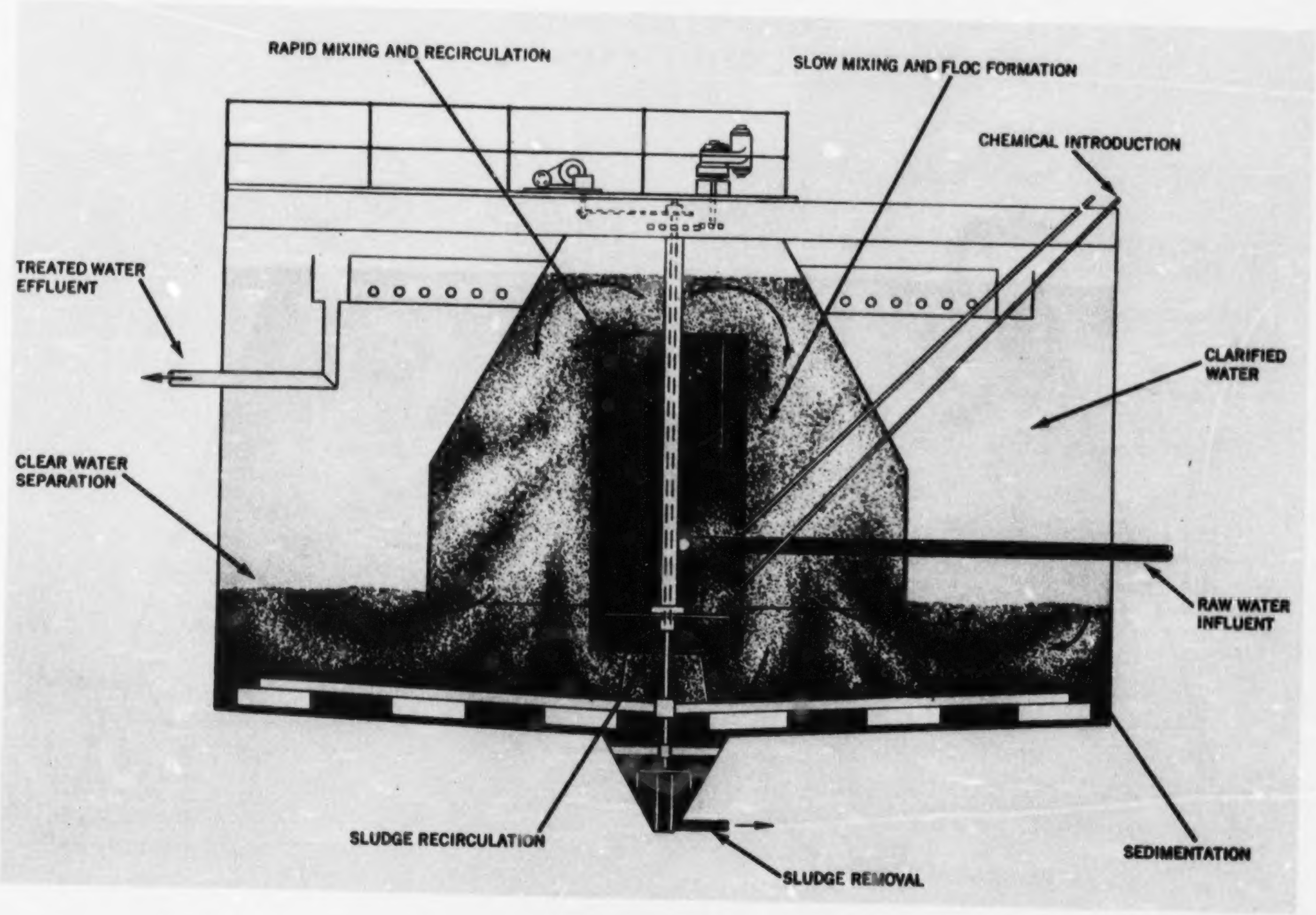
RAPID MIXING AND RECIRCULATION
SLOW MIXING AND FLOC FORMATION
CHEMICAL INTRODUCTION
TREATED WATER EFFLUENT
CLARIFIED WATER
CLEAR WATER SEPARATION
RAW WATER INFLUENT
SLUDGE RECIRCULATION
SLUDGE REMOVAL
SEDIMENTATION

Figure 6-5
SOLIDS CONTACT CLARIFIER WITHOUT SLUDGE BLANKET FILTRATION
(Courtesy of Graver Water Conditioning Co.)

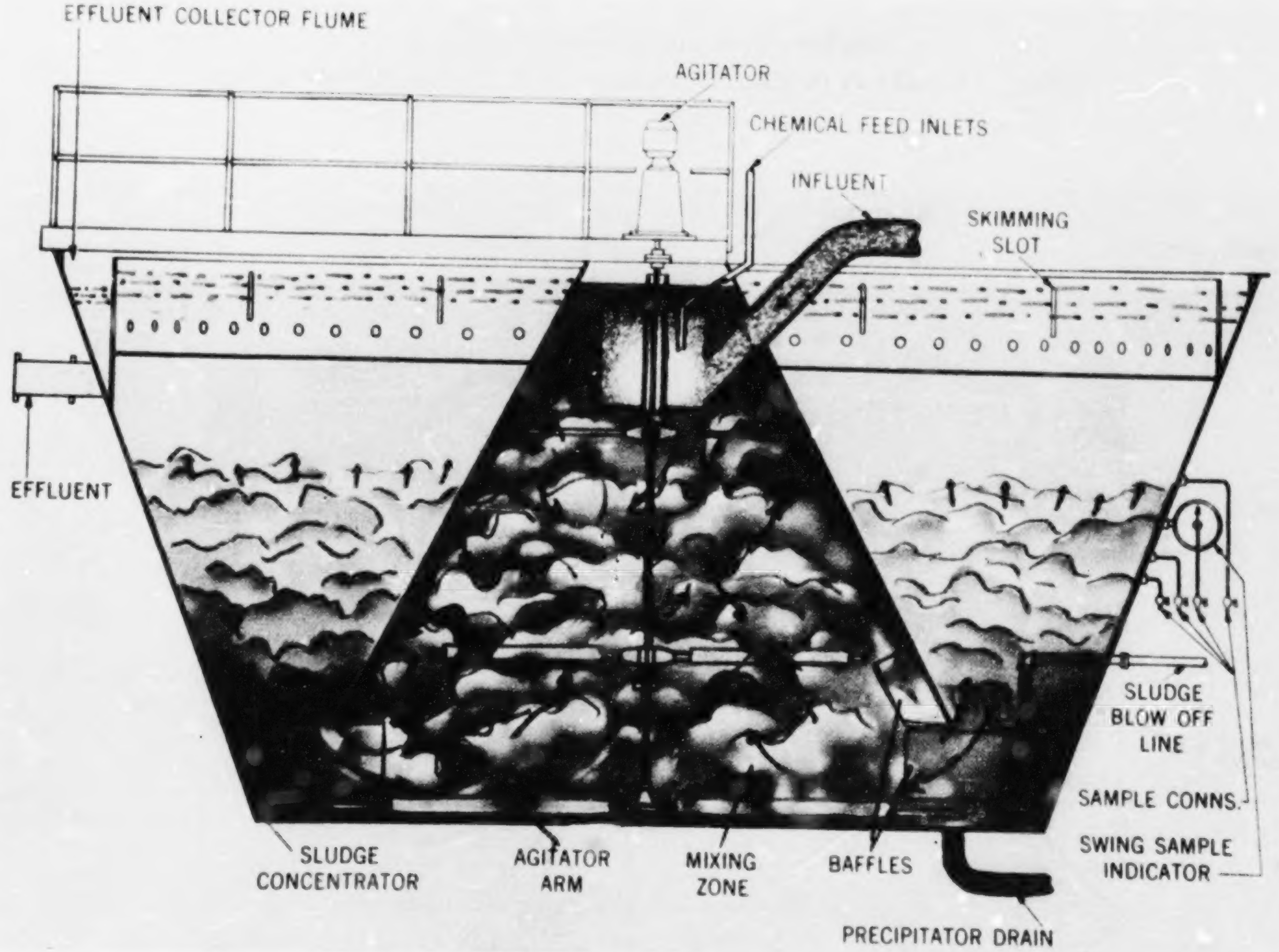
EFFLUENT COLLECTOR FLUME
AGITATOR
CHEMICAL FEED INLETS
INFLUENT
SKIMMING SLOT
EFFLUENT
SLUDGE BLOW OFF LINE
SAMPLE CONNS.
SWING SAMPLE INDICATOR
SLUDGE CONCENTRATOR
AGITATOR ARM
MIXING ZONE
BAFFLES
PRECIPITATOR DRAIN

Figure 6-6

SOLIDS CONTACT CLARIFIER WITH SLUDGE BLANKET FILTRATION

(Courtesy of the Permutit Co.)

Solids-contact clarifiers have been used for the treatment of secondary and primary effluents, as well as for the treatment of raw, degritted wastewater. Lime as the treatment chemical has been used with overflow rates from 1200 to 1700 gpd/sq ft in solids-contact units, while iron compounds and alum have been used at lower values, usually between 500 and 1000 gpd/sq ft. All of these rates come from pilot studies of less than 1 MGD capacity, and may be subject to change at a larger scale due to differences in hydraulics. Polymer treatment can also influence the choice of overflow rates for design if their cost can be economically justified when compared to the cost of lower overflow rates. Detention times in these solids-contact basins have ranged from just over one to almost five hours. Sludge removal rate is dependent on the solids concentration of the underflow, which is a function of the unit design as well as the chemical employed. These pilot plants have reported lime sludge drawoffs from 0.5 to 1.5 percent of the wastewater flow at concentrations of from 3 to 17 percent solids. Alum and iron sludges have not been monitored extensively, but drawoffs have been reported to be 1 to 6 percent of the flow with 0.2 to 1.5 percent solids.

Much of the design information necessary for solids-contact clarifiers has been obtained from water treatment experience. This is not surprising in that the principles of treatment are identical. The characteristics of the solids that are formed and separated are the source of differences. The organic matter contained in the chemically-created sludges causes the sludge to become lighter and also more susceptible to septicity due to the action of microorganisms. The former condition suggests lower hydraulic loadings, while the latter suggests higher ones, given a set physical design. Since sludge septicity is neither universal nor uncontrollable, a lower design overflow rate may comprise much of the necessary adjustment to waste treatment conditions from those of water treatment. As indicated previously, design overflow rates from 1200 to 1700 gpd/sq ft for lime treatment and from 500 to 1000 gpd/sq ft for alum or iron treatment have been successful at less than 1 MGD capacity. Good engineering practice would suggest designing for the lower values cited in order to provide increased flexibility and reliability in the plant. Cold weather peak flow conditions will probably constitute the limiting condition, as water treatment practice has shown that overflow rates are reduced by as much as 50 percent at near-freezing temperature (11). Wastewater will probably not reach such low temperatures in most areas, but the effects are significant.

Design features of solids-contact clarifiers should include:

a. Rapid and complete mixing of chemicals, feedwater and slurry solids must be provided.

b. Mechanical means for controlled circulation of the solids-slurry must be provided with a reasonably wide range of speeds. The maximum peripheral speed of mixer blades should not exceed 5 ft /sec.

c. Means should be provided for measuring and varying the slurry concentration in the contacting zone.

d. Sludge discharge systems should allow for easy automation and variation of volumes discharged.

e. Sludge-blanket levels must be kept a minimum of 5 feet below the water surface.

f. Effluent launders should be spaced so as to minimize the horizontal movement of clarified water.

Tesarik (10) has derived the relationship that the minimum depth below the surface should be one-half the horizontal distance between launders. If the 5-foot minimum sludge-blanket depth (requirement e) is maintained, 10-foot spacing of the effluent weir should not be significantly exceeded. Most of the above requirements are based on those cited in *Water Treatment Plant Design* (1) Further considerations include skimmers and weir overflow rates. Skimmers will probably be necessary when treating raw wastewaters due to the likely presence of floatable solids and grease. Tertiary systems may possibly avoid the use of these devices, but it may be good practice to include them in the design in case anaerobic conditions develop. Weir overflow rates should conform to the requirements of other clarification units.

6.4 References – Chemical Processes

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3. O'Melia, C.R., "The Coagulation Process: Application of Research to Practice", Report submitted to ASCE (Oct. 1969).
4. Hudson, H.E., Jr., and Wolfner, J.P., "Design of Mixing and Flocculating Basins", Jour. AWWA, 59, 1257 (Oct. 1967).
5. Camp, T.R., "Flocculation and Flocculation Basins", Trans. ASCE, 120, 1 (1955).
6. Ives, K.J., "Theory of Operation of Sludge Blanket Clarifiers", Proc. ICE (Br.), 39, 243 (Feb. 1968).
7. Culp, R.L., and Culp, G.L., Advanced Wastewater Treatment, Van Nostrand Reinhold Company, New York (1971).
8. Aitken, I.M.E., "Reflections on Sedimentation Theory and Practice - Part I," Eff. and Water Treatment Jour. (Br.), 7, 4, 226 (Apr. 1967).
9. Burns, D.E., and Shell, G.L., "Physical-Chemical Treatment of a Municipal Wastewater Using Powdered Activated Carbon", Final Report of EPA-WQO Contract No. 14-12-585 (Apr. 1971).
10. Tesarik, I., "Flow in Sludge-Blanket Clarifiers", Jour. SED-ASCE, 93, 105 (Dec. 1967).
11. Eaton, C.D., "Influence of Water Temperature on Performance of Solids-Contact Units", TAPPI, 47, 10, 188A (Oct. 1964).

CHAPTER 7

GRAVITY SYSTEMS

7.1 General

Gravity separation of solids is based on the difference between the specific gravity of the liquid (continuous phase) and the particulate matter (discontinuous phase). Obviously, only two possibilities exist for separation. Heavier than water particulates will settle to the bottom of the liquid and lighter than water particulates will float to the top. Sedimentation is the simplest and most widely employed waste treatment process. An idealized sedimentation basin is shown in Figure 7-1. The basin is divded into four zones, inlet, outlet, settling and sludge. The inlet zone must dissipate the kinetic energy of the feed stream and provide a uniform distribution of the flow entering the settling zone without creating excessive turbulence. The relative importance of proper inlet conditions as compared to outlet conditions has been shown (1). The outlet zone is less critical to basin performance, but low approach velocities to effluent weirs must be maintained for maximum sedimentation efficiency. The difficulties in obtaining optimum inlet and outlet conditions vary with basin shape and flow patterns (1).

The idealized rectangular basin shown in Figure 7-2 illustrates the situation which occurs when a perfect inlet distribution of discrete particles occurs with no particle interaction and no resuspension of particles from the sludge zone. The resultant vector on the discrete particle is then made up of its theoretical settling velocity, usually described by Stokes' Law, and the superficial uniform velocity of the liquid. Numerous theoretical derivations have been made on the assumptions of this diagram. The most notable is that of Camp (2). In reality, uniform flow has been unattainable in the large tanks normally designed. Short-circuiting, turbulence and density currents have been cited as causing the major deviations from the ideal situation. Tracer studies have illustrated the extent of short-circuiting in conventional basins. Most studies of this type have indicated that both vertical flow and long horizontal tanks have better flow characteristics than circular tanks, particularly those of the center-feed type (1). A salt-tracer study of four types of tanks produced the results shown in Figure 7-3 (3).

The sludge zone serves the two-fold function of retaining the solids so that a minimum of resuspension occurs and providing sufficient time for compaction, thereby minimizing sludge pumping requirements. In most cases mechanical scrapers are utilized to slowly move the sludge to the pump drawoff area. An alternative device uses suction to remove the sludge at its deposition point, thereby eliminating the need for displacement prior to removal. A number of different tank configurations are illustrated in Figures 7-4 through 7-6.

7.2 Primary Sedimentation

Primary sedimentation is the most widely used waste treatment process, whether alone or preceding a secondary treatment process. Primary clarifiers are designed to remove the settleable solids and their associated organic load from the wastewater. The basis of design is the overflow rate, usually expressed in gallons/day/square foot (gpd/sq ft), which is equal to the flow in gallons/day divided by the surface area of the clarifier in square feet. It is obvious that overflow rate in itself does not totally determine the efficiency of a tank. In

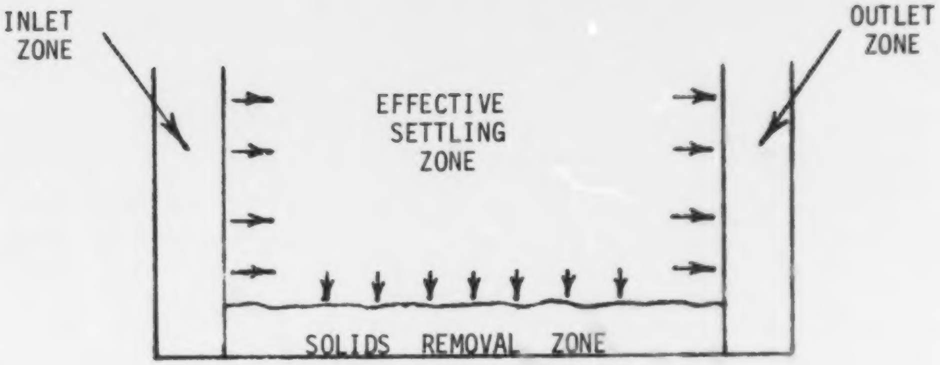
INLET ZONE
OUTLET ZONE
EFFECTIVE SETTLING ZONE
SOLIDS REMOVAL ZONE

Figure 7-1

FUNCTIONAL ZONES IN AN IDEALIZED SEDIMENTATION BASIN

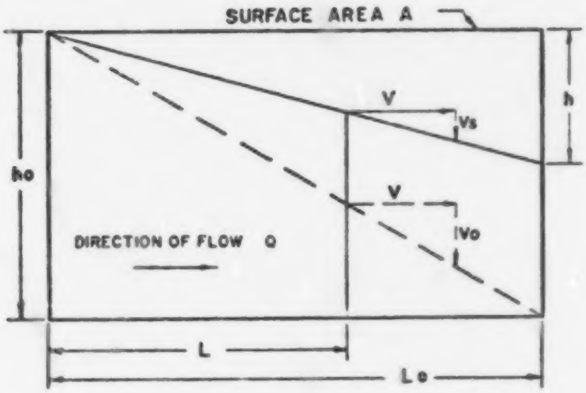
SURFACE AREA A
V
Vs
h
ho
V
Vo
DIRECTION OF FLOW Q
L
Lo

Figure 7-2

IDEALIZED SETTLING PATHS OF DISCRETE PARTICLE IN A HORIZONTAL FLOW TANK

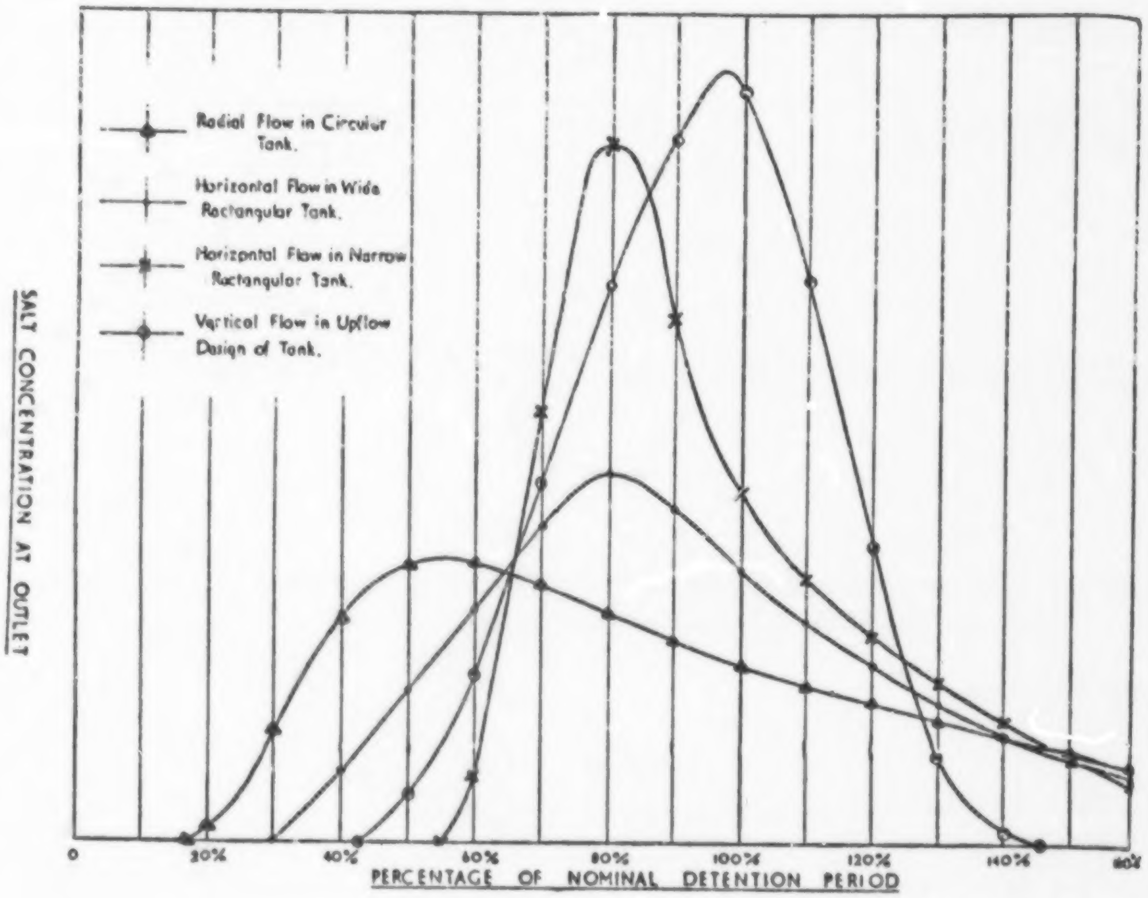

Figure 7-3

RESULTS OF SALT-INJECTION TESTS WITH DIFFERENT TYPES OF SEDIMENTATION TANKS

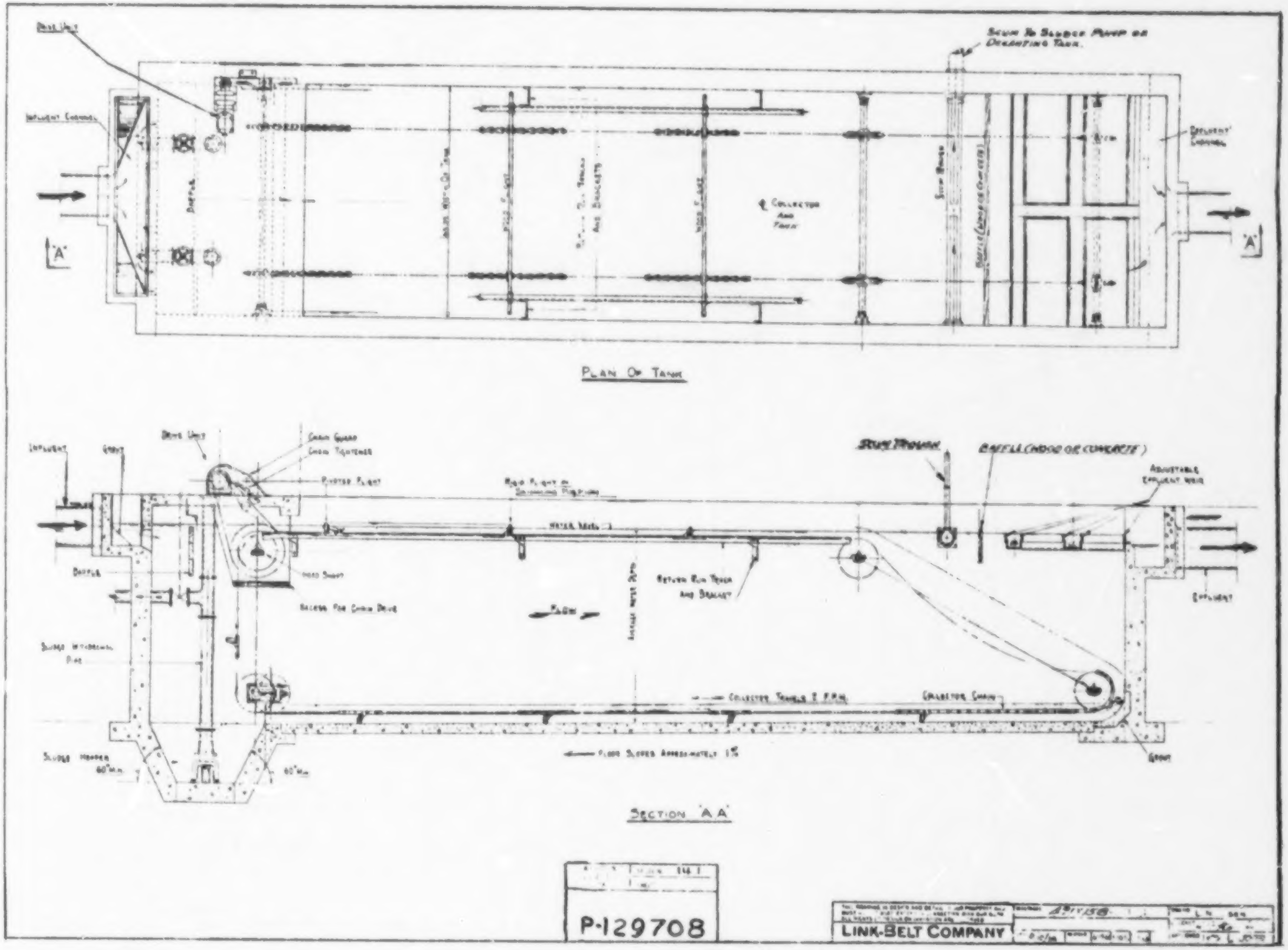
Drive Unit
Influent Channel
Baffle
Scum to Sludge Pump or Decanting Tank.
Effluent Channel
Return Run Trough and Brackets
℄ Collector and Tank
Scum Board
Plan Of Tank
Influent
Grout
Drive Unit
Chain Guard
Chain Tightener
Pivoted Flight
Rigid Flight in Skimming Position
Water Level
Head Shaft
Recess for Chain Drive
Flow
Return Run Track and Bracket
Scum Trough
Baffle (Wood or Concrete)
Adjustable Effluent Weir
Effluent
Baffle
Sludge Withdrawal Pipe
Sludge Hopper 60° Min.
60° Min.
Collector Travels 2 F.P.M.
Collector Chain
Floor Sloped Approximately 1%
Grout
Section 'A-A'
P-129708
Link-Belt Company

Figure 7-4
RECTANGULAR SETTLING BASIN WITH SLUDGE AND SCUM COLLECTOR

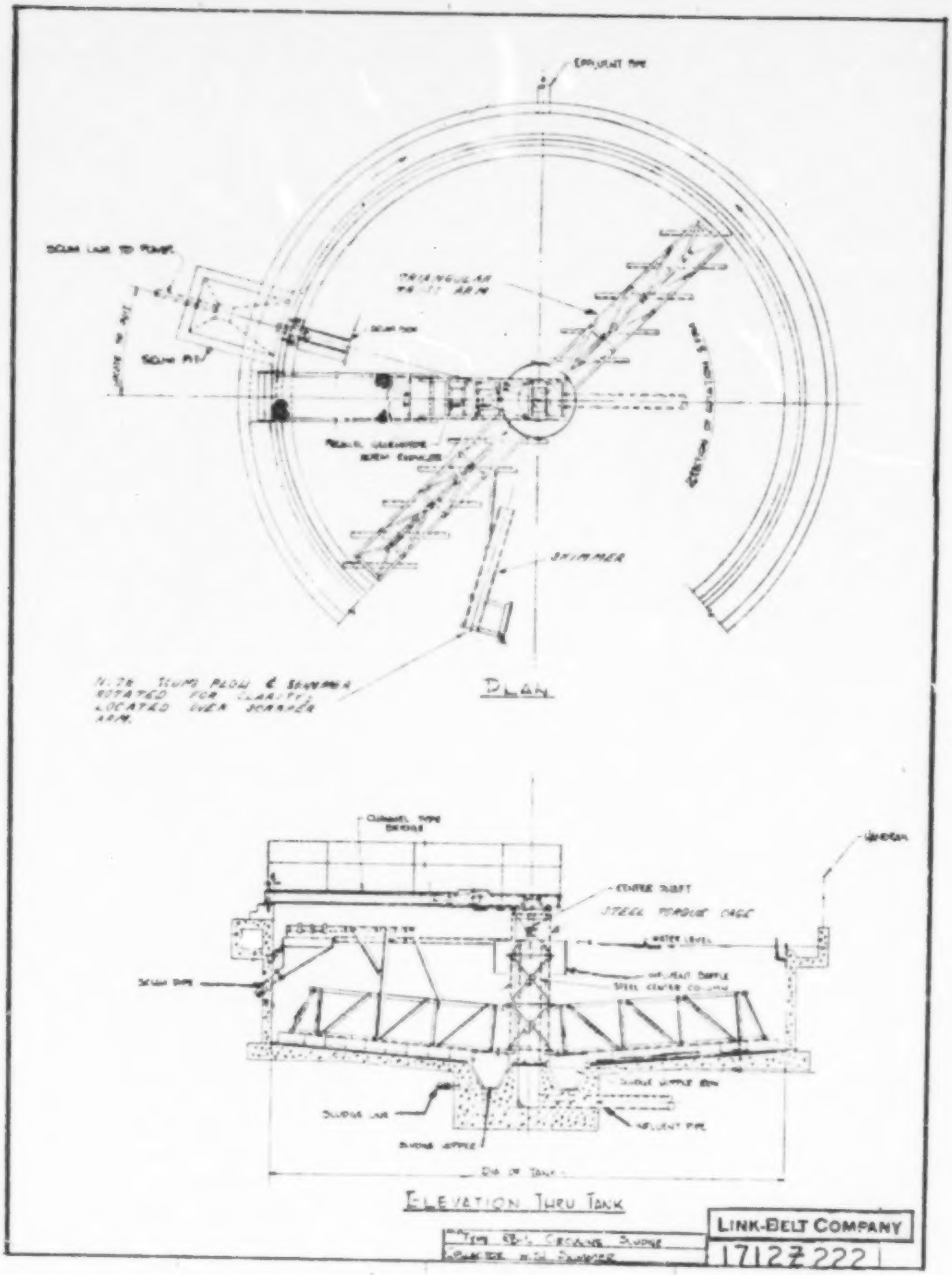
EFFLUENT PIPE
SCUM LINE TO PUMP
SCUM PIT
TRIANGULAR TRUSS ARM
SKIMMER
NOTE SCUM PLOW & SKIMMER ROTATED FOR CLARITY; LOCATED OVER SCRAPER ARM.
PLAN
CHANNEL TYPE BRIDGE
HANDRAIL
CENTER SHAFT
STEEL TORQUE CAGE
WATER LEVEL
INFLUENT BAFFLE
STEEL CENTER COLUMN
SCUM PIPE
SLUDGE LINE
INFLUENT PIPE
DIA OF TANK
ELEVATION THRU TANK
LINK-BELT COMPANY
17127222

Figure 7-5
CIRCULAR SETTLING BASIN – CENTER FLOW

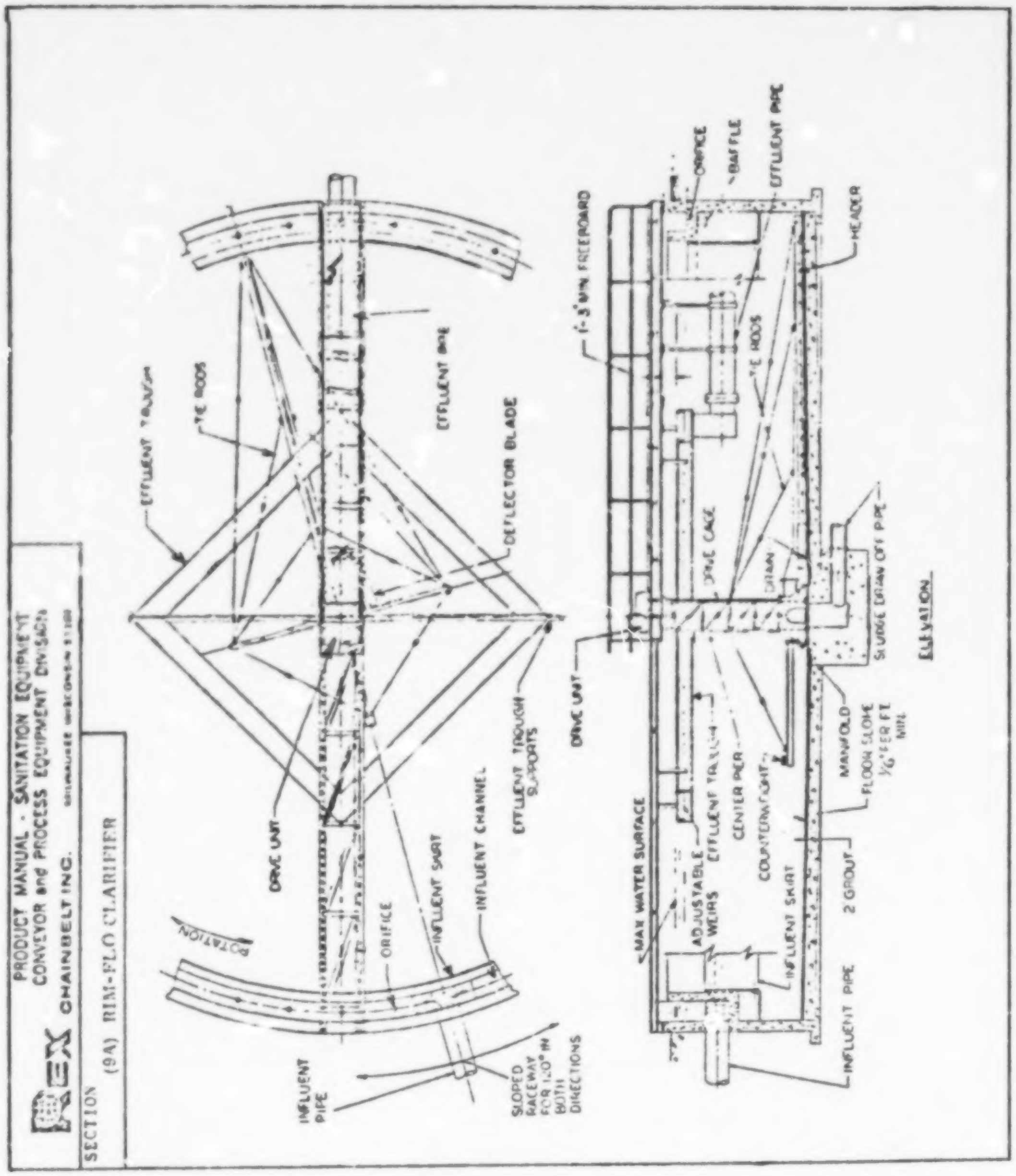
PRODUCT MANUAL - SANITATION EQUIPMENT
CONVEYOR and PROCESS EQUIPMENT DIVISION
REX CHAINBELT INC.
MILWAUKEE WISCONSIN 53201
SECTION
(9A) RIM-FLO CLARIFIER
EFFLUENT TROUGH
TIE RODS
EFFLUENT PIPE
DEFLECTOR BLADE
EFFLUENT TROUGH SUPPORTS
DRIVE UNIT
INFLUENT SKIRT
INFLUENT CHANNEL
ORIFICE
ROTATION
INFLUENT PIPE
SLOPED RACEWAY FOR 120° IN BOTH DIRECTIONS
1'-3" MIN FREEBOARD
ORIFICE
BAFFLE
EFFLUENT PIPE
TIE RODS
HEADER
DRIVE CAGE
DRAIN
SLUDGE DRAW OFF PIPE
ELEVATION
DRIVE UNIT
MAX WATER SURFACE
ADJUSTABLE WEIRS
EFFLUENT TROUGH
CENTER PIER
COUNTERWEIGHT
INFLUENT SKIRT
MANIFOLD
FLOOR SLOPE 1/16" PER FT MIN
2" GROUT
INFLUENT PIPE

Figure 7-6
RIM FLOW SETTLING BASIN

addition to the previously noted influent and effluent zone effects, the basin shape and size as well as the settling characteristics of each individual wastewater will influence the solids removal efficiency of a given sedimentation tank. Seasonal temperatures can also exert a significant influence on basin performance.

The Ten-State Standards recommend an overflow rate of 600 gpd/sq ft or less for plants having a design flow of less than 1MGD, but allow higher rates for larger plants (4). These standards also require a minimum depth of 7 feet for primary tanks with mechanical cleaning and a minimum length from inlet to outlet of 10 feet. Smith (5) has discussed the use of excess capacity factors in design. American practice generally utilizes depths from 7 to 12 feet, which, when combined with overflow rate, provide nominal detention periods from 1 to 2 hours (6). European practice also considers mean flow velocities with a design value for rectangular basins of about 2 feet/minute (fpm). This type of basin is designed with depth to length ratios of 1/10 to 1/30. Widths generally vary from 20 to 30 feet, and depths from 5 to 10 feet (7). Details of more subtle design considerations such as tank slopes and ancillary devices can be found in the Water Pollution Control Federation (WPCF) Manual of Practice No. 8 (6). Tube settlers, discussed elsewhere, are often installed in existing sedimentation basins to increase total throughputs.

As previously noted, the removal efficiency of primary sedimentation facilities will vary widely. WPCF MOP No. 8 (6) shows a nearly straight line inverse relationship between overflow rate and suspended solids removal, while the 10-State Standards show a similar relationship for BOD removal (4). Fair and Geyer indicate ranges of 40 to 70 percent and 25 to 40 percent removal for suspended solids and BOD, respectively (8). Kalbskopf and Metcalf and Eddy show a family of curves for different influent suspended solids concentrations, the former versus horizontal velocity and the latter versus detention time (7),(9). The values of all these correlations fall in ranges of 35 to 75 percent and 25 to 40 percent removal of suspended solids and BOD, respectively. Sludge volumes are generally less than 1 percent of the total flow, and solids concentrations are usually between 2 to 7 percent of the underflow. Skimmings are cited as 0.1 to 0.7 cu ft/MG (7).

7.3 Secondary Sedimentation

The principles governing design of secondary sedimentation tanks are significantly different from those used for primary clarifiers. The major reason for the difference lies in the amount and nature of the solids to be removed. While primary settlers are designed on the basis of overflow rate alone, secondary clarifiers must be designed on the basis of overflow rate and solids loading. The greater concentration and lighter nature of the mixed-liquor suspended solids requires that the underflow concentration be considered in design. Settling rates are slower, as hindered settling prevails instead of free settling which occurs in primary basins. Solids loading, expressed as pounds of dry solids/day/square foot of surface area, has been discussed in theoretical terms by Dick (10) and others (11). The information needed for an accurate determination can be obtained from laboratory settling tests. After the limiting solids loading has been determined on the basis of desired underflow concentration, the required area of basin can be computed using design flow and mixed-liquor suspended solids (MLSS) concentration. The area required by this calculation and the area required by the overflow rate must then be compared, with the larger being the design size. A factor which strongly affects the limiting solids loading is the sludge volume index (SVI). As this value increases, sludge settling and concentration become more difficult. Kalbskopf recommends the use of a sludge volume loading parameter which would be the product of the solids loading times the SVI (7).

The 10-State Standards (4) do not consider solids loading, but recommend that overflow rates not exceed 800 gpd/sq ft. They also require a minimum depth of 8 feet for activated sludge systems. Metcalf and Eddy (12) recommend an overflow rate of 1000 gpd/sq ft (less if MLSS exceed 2000 mg/1) along with a depth of 12 feet for clarifiers following a nitrification system. They also suggest a 1200 gpd/sq ft maximum overflow rate for denitrification facilities. Sludge volume storage requirements will result in deeper tanks. In many cases the computed volume of tanks will be multiplied by an excess capacity factor of about 2, including a surface area increase of 25 to 75 percent, to obtain a final design (5).

Inlet requirements are similar to those of primary tanks, although some European designers have indicated the favorability of flocculation facilities in the inlet area (7). The influence of the effluent weir overflow rate is greater in secondary clarifiers than in primary tanks due to the nature of the solids, even though 10-State Standards (4) indicate the same maximum weir loading for both. For plants of less than 1 MGD capacity, it is 10,000 gpd/ft and for larger plants 15,000 gpd/ft. Detention times are generally longer than in the primary, ranging from 2 to 3 hours. Scum removal is generally required for all clarifiers and is usually integrated with sludge removal mechanisms. Activated sludge concentration in secondary clarifier underflow is generally noted to vary from 0.5 to 2.0 percent solids, while trickling filter humus tanks are found to vary from 3 to 7 percent solids in the resulting sludges.

The performance of secondary clarifiers is not generally cited, as they are considered integral to activated sludge systems. However, they can be rated on the basis of effluent quality to some extent. Usually, a large portion of the effluent organic content is associated with the suspended solids. The success of treatment processes such as microscreens in treating secondary effluent must be attributed to the shortcomings of secondary clarifiers. Although no clarifier will completely remove all suspended solids, a properly designed facility which results from the use of some of the available information, such as in the WPCF MOP No. 8, should yield a good product if the biological process is working properly.

7.4 Chemical Sedimentation

Sedimentation of chemically-coagulated or precipitated wastewaters is similar in principle to the other types noted above. Overflow rates from 450 to 1620 gpd/sf have been cited for plants where existing primary facilities were successfully used for sedimentation of iron and polymer coagulated raw sewage (13),(14). Design of sedimentation tanks for sewage coagulation with alum or iron salts, with or without polymer addition, will generally require an average overflow rate of not more than 1000 gpd/sf and a detention time of at least 2 hours (6). The use of lime will probably allow a somewhat higher overflow rate than alum and iron systems since water treatment experience indicates a settleability of these precipitates from one to 4 times greater than alum flocs(15),(16). Allowable weir rates from water treatment are cited to be 11,000 to 29,000 gpd/ft, depending on the nature of the chemical solids (15). Chemical sludges generated from these systems may vary from 0.5 to more than one percent of the throughput volume and have solids concentrations from about 1 to 15 percent, depending on the chemical used and basin efficiency. Materials of construction for tanks, piping, and other associated devices will require consideration of the chemicals used, e.g., glass-lined or PVC pipes for lime sludge drawoff.

7.5 Flotation

In essence, flotation is a unit process whereby particulate matter is separated from a wastewater by causing it to float to the liquid surface. The density difference which drives this separation is artificially induced by the addition of a third phase, a gas, to the system. The gas, in the form of bubbles, becomes attached to the solid particulates to form a gas-solid aggregate which has an overall bulk density less than the liquid, with resultant rise of the aggregate to the surface of the fluid.

As a primary clarification device, flotation has been demonstrated to remove from 40 to 65% of the influent suspended solids and their associated BOD (18),(19). Although it has been suggested that flotation units can serve as a grit removal device in its primary clarification position, no data have been found to support an application of this concept (20),(21),(22). The most obvious application of flotation in primary treatment is in the expansion of clarification capacity by conversion of existing clarifiers as an alternative to additional construction of more sedimentation basins (19),(22). No chemicals were utilized in any of these installations.

The inherent quality of short detention periods for flotation units suggests that they should be advantageous as secondary clarifiers. The dearth of data for this application appears to point out that effluent quality is inferior to sedimentation, but highly concentrated sludges (up to 10% solid) can be obtained. These results were obtained with units operating without chemicals (22), (23).

As to the method of air dispersal in the liquid, flotation can be classified as dispersed-air or dissolved-air. The former method involves introduction of air by mechanical means directly into the flotation unit. For various reasons, discussed in the literature, this method is not well-suited for municipal waste treatment operations. It is, however, widely used in the mining industry and for some industrial waste applications where particularly high solids concentrations exist. Dissolved-air flotation is the most commonly used for waste treatment. Pressure systems are generally preferred over vacuum types. In these systems influent or recycled effluent is pressurized to 25 - 70 psig with concurrent addition of 5 to 8% air by volume. The pressurized mixture is then held for a short period in a retention tank to allow for effective gas transfer to the liquid and to eliminate excess or undissolved air. Subsequently, the pressurized mixture passes through a pressure-regulating device which drops the pressure to atmospheric prior to introduction to the flotation chamber. Wastewater flotation generally involves pressurization of recycled effluent to avoid solids destruction. Influent is blended with this stream following pressure release. Since pressure flotation is widely used, the design of the ancillary equipment has been thoroughly discussed in the literature (24),(25),(26).

The inlet design of the flotation tank is critical to the proper performance of that unit. Baffles, walls, and other obstructive energy-dissipating devices tend to destroy aggregate bonding with resulting loss in flotation efficiency. Also, turbulence in the region of the froth will result in losses of floated solids. Ettelt (25) reported several different designs of inlet structures in his prototype units. The tangential flow inlet appears to offer considerable promise where such designs are compatible with the entire structure.

The flotation tank is designed to permit aggregate rise with a minimum of interference in the form of turbulence or obstructions and to provide for removal of floated froth, settled sludge and treated effluent. Effluent ports must be sufficiently submerged to prevent interference with the froth on the surface. Sludge removal can be accomplished by gravity as well as mechanical means. Froth removal occurs through the use of skimming devices which carry the froth from its original areal position to the point of disposal. A typical flotation tank is shown in Figure 7-7.

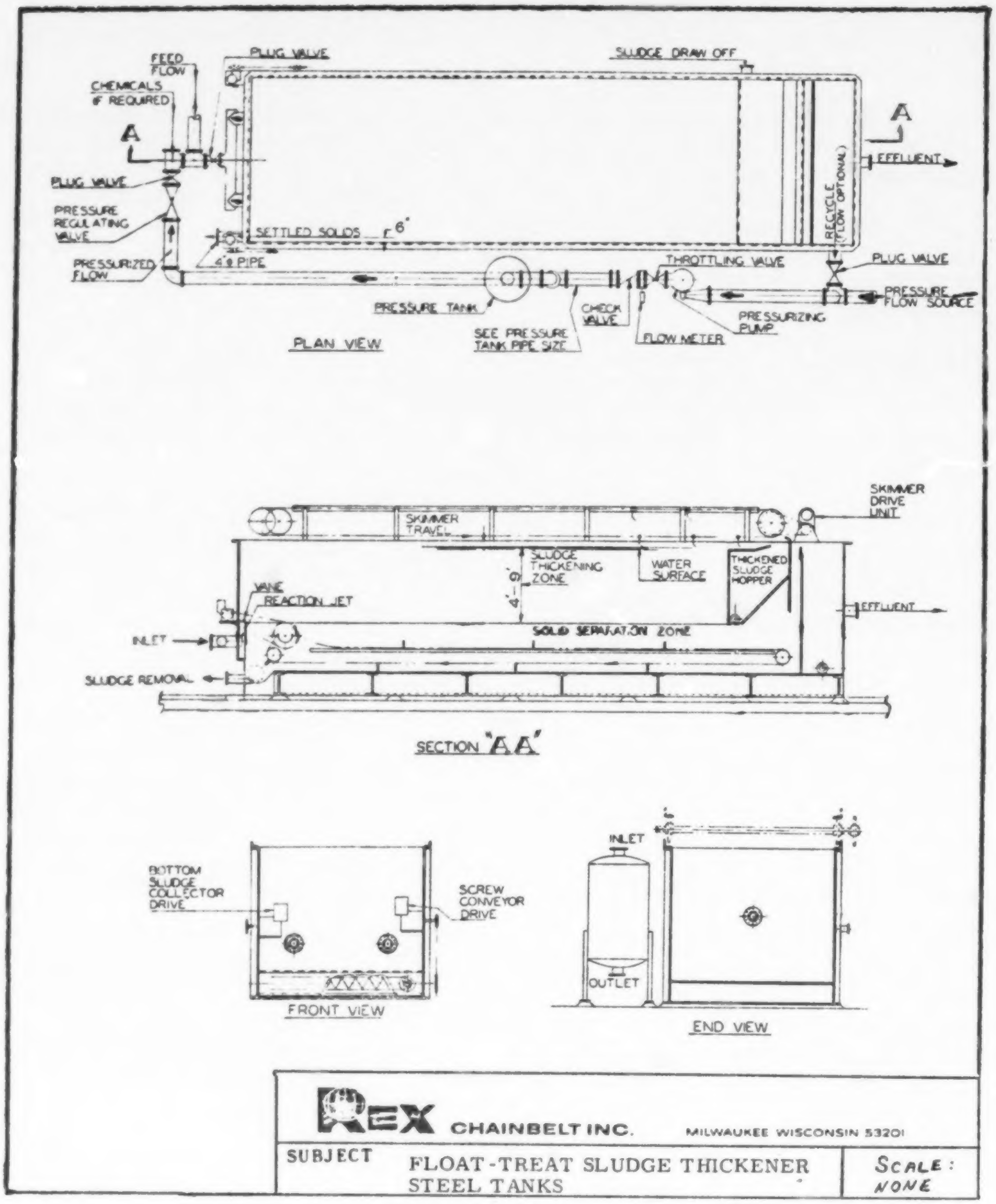
FEED FLOW
PLUG VALVE
SLUDGE DRAW OFF
CHEMICALS IF REQUIRED
A
A
EFFLUENT
PLUG VALVE
PRESSURE REGULATING VALVE
RECYCLE (FLOW OPTIONAL)
SETTLED SOLIDS
6"
4"ϕ PIPE
PRESSURIZED FLOW
THROTTLING VALVE
PLUG VALVE
PRESSURE FLOW SOURCE
PRESSURE TANK
CHECK VALVE
PRESSURIZING PUMP
SEE PRESSURE TANK PIPE SIZE
FLOW METER
PLAN VIEW
SKIMMER DRIVE UNIT
SKIMMER TRAVEL
SLUDGE THICKENING ZONE
4'-9"
WATER SURFACE
THICKENED SLUDGE HOPPER
VANE
REACTION JET
EFFLUENT
INLET
SOLID SEPARATION ZONE
SLUDGE REMOVAL
SECTION "AA"
INLET
BOTTOM SLUDGE COLLECTOR DRIVE
SCREW CONVEYOR DRIVE
OUTLET
FRONT VIEW
END VIEW
REX CHAINBELT INC.
MILWAUKEE WISCONSIN 53201
SUBJECT
FLOAT-TREAT SLUDGE THICKENER STEEL TANKS
SCALE: NONE

Figure 7-7

PRESSURE-FEED FLOTATION SYSTEM

The ranges of design criteria for floation tanks are quoted to be (21),(23),(24),(26),(27),(28):

Overflow Rate:	1-6 gpm/sq ft
Depth:	4-9 ft
Detention Time:	10-40 min

7.6 High-Rate Settlers

It has long been recognized that solids removal by settling could be accomplished within a few minutes if shallow basins could be used,(29),(30), but early prototypes of such equipment were not successful because of problems with flow distribution and sludge removal. Improvements in design have been made, and now two basic types of high-rate settlers are being applied in the treatment of wastewater. These are the tube settlers* which consist of modules of inclined tubes with small hydraulic radii suspended in a basin, and the Lamella Separator** which consists of suspended, inclined plates. Much information on the equipment has been published or supplied by the manufacturers. However, operating data for wastewater treatment application are limited. Satisfactory operation is indicated for most installations, although there are reports of unresolved problems with buildup of slime growths which may constrict the shallow passages through which the water must pass. Periodic cleaning may therefore be necessary and equipment should be designed to permit ready access to the settling units. The Lamella Separator is recommended by its manufacturer for wastewater solids removal only in conjunction with chemical coagulation.

The principal advantages offered by high-rate settlers lie in their compactness. These devices require only a small percent of the space of a conventional gravity settling unit. This difference in space requirements has a number of benefits: a) the high-rate settlers may be less costly to fabricate than a new conventional clarifier; b) the units are useful for upgrading an overloaded plant; c) where additional facilities are required for an existing plant, the high-rate systems have the obvious advantage of requiring considerably less land; and d) for the treatment of industrial wastewaters, it is feasible to place separate units at different locations within the plant to permit optimum product recovery and/or wastewater disposal. Factors such as short circuiting, turbulence and convection currents which adversely affect conventional clarifiers are minimized. In some specific cases mechanical devices such as sludge rakes may be eliminated because of the compact size of the high-rate settler.

7.7 Tube Settlers

In the basic tube settler system, wastewater carrying suspended solids is subjected to clarification by particle sedimentation as it moves from an influent well or distribution chamber, upward through small tubes and into a collection gallery, clearwell or launder. The basic configuration for the installation of tube settlers uses the steeply-inclined (45-60°) tubes. When the tubes are installed in this position, continuous gravity drainage of the settleable solids in the tubes is achieved. The incoming solids settle to the tube bottom and then exit by sliding downward as shown in Figure 7-8. In this flow pattern the solids settling to the tube bottom are trapped in a downward flowing stream of previously settled and concentrated solids. The tube cross section may be circular, square, rectangular, hexagonal or other suitable shape.

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* *Manufactured by Neptune Microfloc, Inc., Graver Water Conditioning Co., Permutit Co. and others.*

** *A product of the Axel Johnson Institute for Industrial Research, Nynashamn, Sweden. Represented in the U.S. by Parkson Corp., Ft. Lauderdale, Fla.*

The theoretical settling pattern of discrete particles in an ideal rectangular basin has been described as straight lines where all particles with identical settling velocities move in parallel paths. A particle being carried forward by the velocity of the liquid flow through the tank must settle at such a minimum rate that it reaches the bottom before the flow leaves the tank. Thus, if V_l is the velocity of the fluid, V_s the settling velocity of the particle, L the length of the tank, and d its depth, then a particle at the influent will settle to the bottom of the device only if:

$$V_l \max = V_s L/d$$

This means that the flow velocity at which a horizontal clarifying device may be operated successfully is directly proportional to the length of the device, and inversely proportional to the depth. Since this analysis applies to horizontal tubes and plates as well as horizontal tanks, and height of the tubes is usually a few inches as compared to a height of several feet in a horizontal tank, distances and required settling times can be dramatically reduced provided that flow is kept laminar. Shallow settling devices become self-flushing if they are inclined at an angle which exceeds the angle of repose of the settled material. When the device is inclined, the particle no longer falls through distance d but some longer distance d' as depicted in Figure 7-9.

Clearly, d' is related to d by the relation

$$d' = \frac{d}{\text{Cos } \Theta}$$

where Θ is the angle the device is inclined from the horizontal plane. In a case where

$$\Theta = 60^{\circ}, \frac{1}{\text{Cos } 60^{\circ}} = 2$$

or the settling distance is twice the height of the device. The equation for the case of inclined clarifying devices becomes:

$$V_l \max = \frac{V_s L}{d/\cos\Theta} = \frac{V_s L \cos\Theta}{d}$$

The above equation is limited to angles between 0° and $\tan^{-1}\Theta = \frac{L}{d}$.

Laminar flow is necessary for the efficient and effective operation of a tube settler because the random connective currents present in turbulent flow redistribute any solids that tend to settle. Laminar flow is maintained at high flow rates in small tubes by the increased "drag" effect of the relatively large surface of the tubes. A purely theoretical analysis of high rate settling has been done (32).

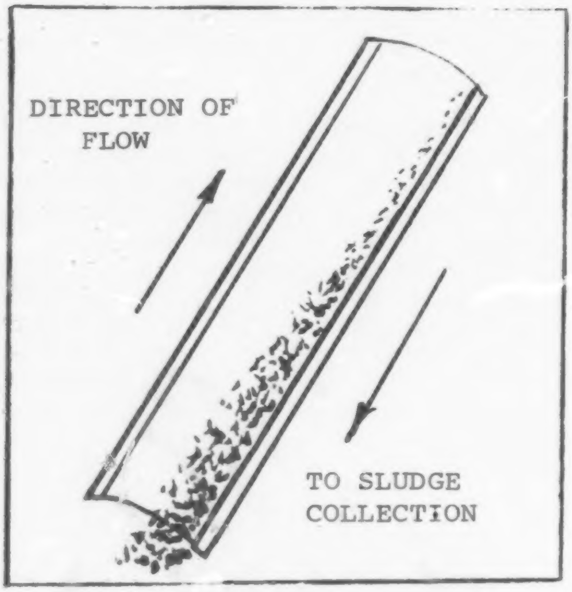
DIRECTION OF FLOW
TO SLUDGE COLLECTION

Figure 7-8
TUBE SETTLERS – FLOW PATTERN

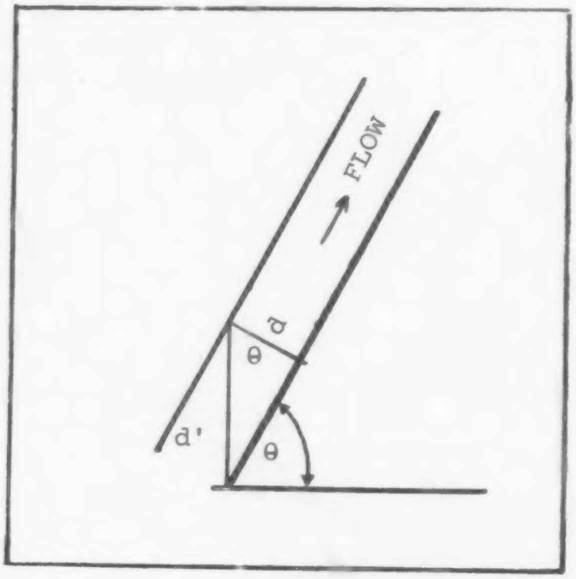

Figure 7-9
INCLINED TUBE

Tests to determine the optimum angle of inclination for tubes showed that tube efficiency at 60° was comparable to that obtained at 5° (31). It appeared that, when the angle of inclination exceeded the angle of repose of the settled sludge, additional flocculation occurred as the heavier floc settled and collided with the smaller, upward moving floc, contributing to increased efficiency. A continuing increase in angle eventually results in the tube acting as an upflow clarifier.

For large scale plants where coagulation is to be used for clarification, steeply inclined, self-cleaning settling tube modules, similar to the one shown in Figure 7-10 can be readily incorporated into new clarifier designs in order to reduce areal requirements. The size of the tube openings and tube length can be varied depending upon the particular application. The most significant application of the tube clarifier concept is its application to secondary clarification. Figure 7-11 shows how modules of steeply inclined tubes can be placed in either a circular or rectangular secondary clarifier. The activated sludge mixed liquor or trickling filter effluent is introduced beneath the tube modules. The flow is upwards through the modules with the solids falling from the tube bottoms into the sludge collection zone beneath the tubes. The clarified effluent is collected above the tube modules. Secondary clarification has been provided in basins, as shown in Figure 7-12, at surface loading rates as high as 4,000 gpd per square foot, or about five times the loading rates used with conventional basins.

Tube modules can be installed in an existing secondary clarifier to increase its capacity. An installation of this type was carried out early in 1968 near Pittsburgh, Pa. In this case, the secondary clarifier in a 130,000 gpd contact stabilization plant was allowing 50-80 mg/l suspended solids to be carried into the plant effluent. Tube modules were installed over half of the clarifier surface with the remainder of the clarifier removed from service. The plant effluent now contains only 5-30 mg/l suspended solids even though the surface loading on the modified basin now approaches 3,000 gpd per square foot based on the total daily flow. Other applications of tube settlers to wastewater treatment have been published (33),(34). Results obtained with tube settler installations on wastewater are summarized in Table 7-1.

7.8 Lamella Separator

The Lamella Separator shown in Figure 7-13 consists of a nest of parallel inclined plates through which the suspension is passed, each plate having an effective settling area equal to its projection onto a horizontal plane. By putting the plates in very close proximity, it is possible to obtain a high settling capacity in a very small volume. (The term "Lamella" refers to the liquid layer between adjacent plates.) The basic difference between the Lamella Separator and tube settling units, which also utilize the principle of multiple inclined surfaces, is that the Lamella Separator is fed from the top whereas the tubes are fed from the bottom. Flow of the liquid is thus cocurrent with the sludge, so that the frictional drag of the liquid and the force of gravity work in conjunction to transport the sludge down the plates. In addition, with the liquid and sludge flowing in the same direction, less shear at the interface and less probability for re-entraining sludge has been claimed.

Although the basic concept of the Lamella Settlers is not new, a number of problems had to be overcome before the idea could be reduced to practice. Of particular importance, the possibility of secondary flows has been reduced by the use of flow stabilizing devices in the inlet to the lamella and by dividing the unit into many compartments. The exit of the clarified effluent is accomplished by a tube network which collects the liquid uniformly

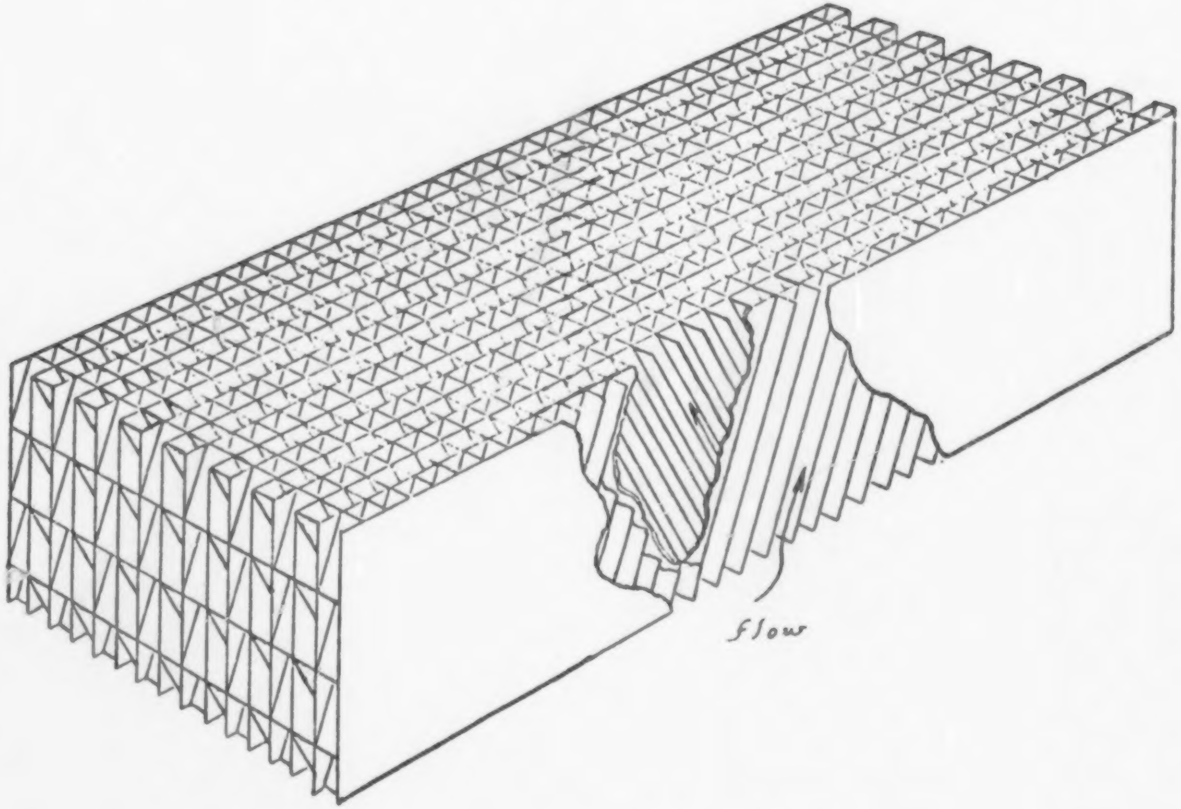

Figure 7-10
MODULE OF STEEPLY INCLINED TUBES
(Courtesy Neptune Microfloc, Inc.)

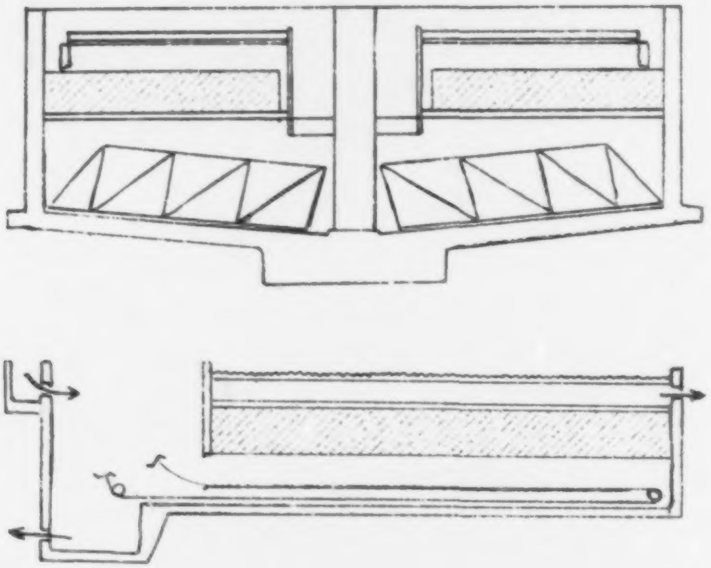

Figure 7-11

TUBE SETTLERS IN EXISTING CLARIFIER

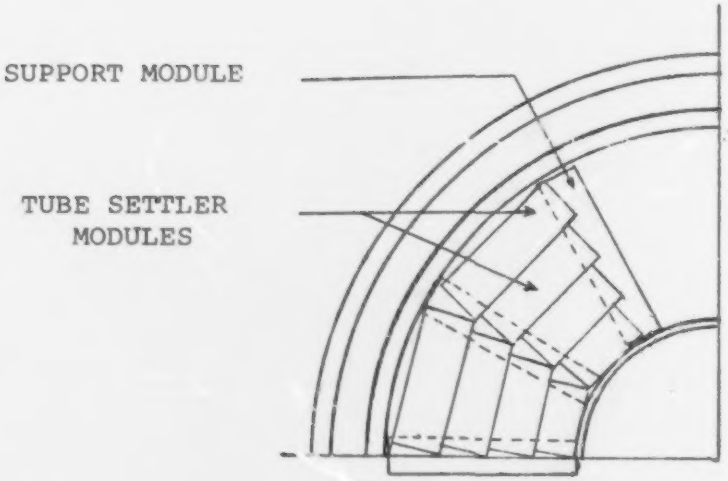
SUPPORT MODULE
TUBE SETTLER MODULES

Figure 7-12

PLAN VIEW OF MODIFIED CLARIFIER

Table 7-1

TUBE SETTLER INSTALLATIONS

				TUBE DATA		EXISTING FACILITY	
PLANT LOCATION	TYPE	SIZE MGD	TUBE LOC.	O.R. (gpm/sf)	EFF. SS (mg/l)	O.R. (gpm/sf)	EFF. SS (mg/l)
Philomath, Oregon	TF[a]	0.15	Aerator[a]	3.3	43	–	–
Philomath, Oregon	TF	0.15	S. Clar.	3.3-4.6	60-70	0.6	60-70
Philomath, Oregon	TF	0.15	P. Clar.	2.1-3.3	34-41[b]	0.84	40-45[b]
Hopewell Twp., Pa.	AS	0.13	S. Clar.	2 - 3	27	0.34	60-70
Miami, Fla.	AS	1.0	S. Clar.	1.7	33	1.3	500
Ontario, Canada	AS	–	Aerator	1.0	28	–	–

[a] Secondary clarifier converted to aerator + tubes

[b] Percent removal rather than concentration

across the width of the plate, with return through a center tube to prevent recontaminating or disturbing the liquid being clarified. These features are shown in Figure 7-14.

A Lamella Separator consists of one or more modules, each consisting of perhaps 10-100 plates. The plates are typically 1.5 m wide x 2.5 m long, are splaced 25-50 mm apart, and are inclined at 25-45° to the horizontal. The total number of plates required can be determined by laboratory settling tests. Materials of construction can be of many varieties, but where temperature conditions allow, PVC is usually the best material to use for the plates from both a technical and an economic standpoint.

The Lamella Separator is being used in Sweden at several locations to clarify lime-treated municipal wastewaters. One unit operating at 0.8 MGD removes 96 percent of the solids applied to it. The Lamella Separator has been successfully tested on a pilot scale for clarification of potable water, steel pickling rinse waters and pulp and paper mill effluents.

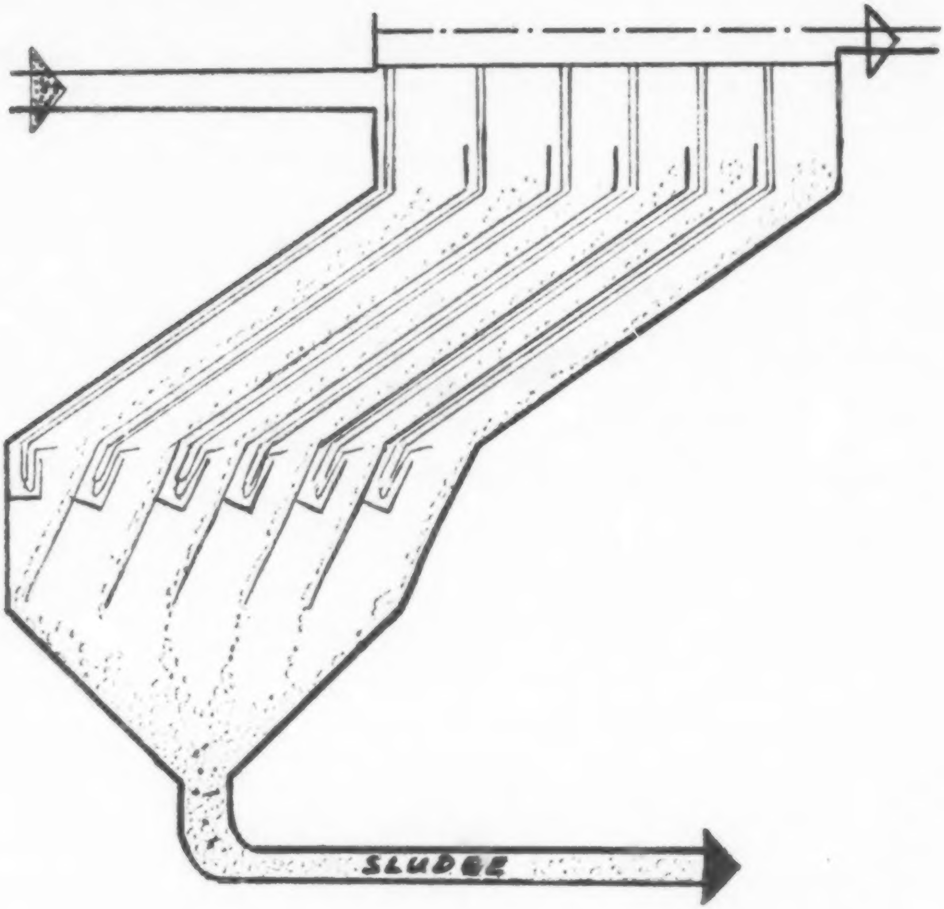

Figure 7-13
LAMELLA SETTLER

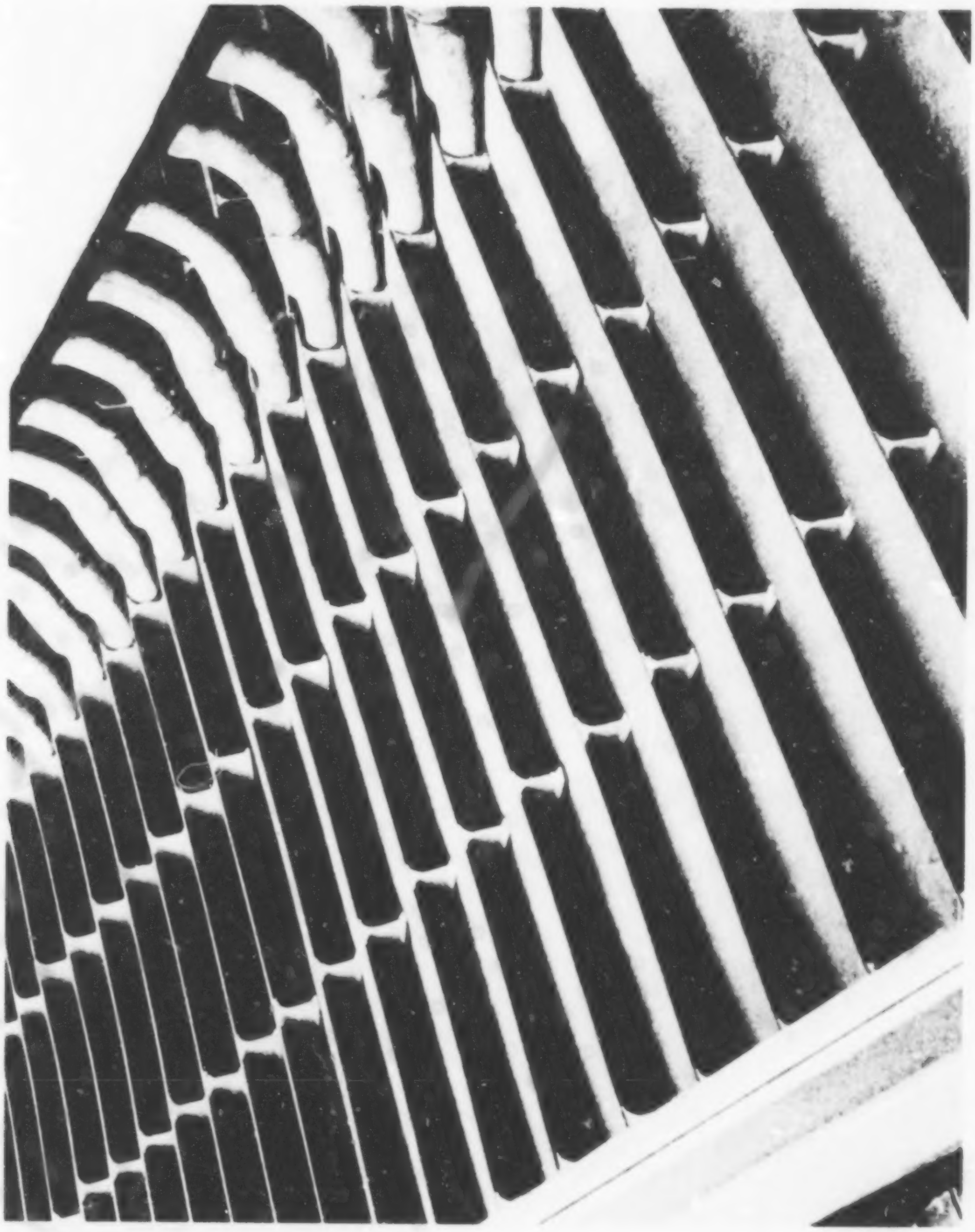

Figure 7-14

LAMELLA SEPARATOR (From Above)

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CHAPTER 8

PHYSICAL STRAINING PROCESSES

8.1 General

Physical straining processes are defined for the purposes of this manual as those processes containing elements which remove solids by virtue of physical restrictions at their surface and which have no appreciable thickness in the direction of the liquid flow. These restrictions may be from the straining device itself or may be due to a thin layer of previously-removed solids deposited upon a relatively coarse substrate or fabric. Processes which fit this category are rotary screens, vibrating screens, ultrafiltration and diatomaceous earth filters, among others. Since the purpose of this manual is to provide information which will be of value in designing or modifying solids removal facilities, only processes on which substantial reliable information of previous use is available will be discussed.

8.2 Microscreening

Microscreening has been a viable solids removal process for twenty years. Its use as a tertiary unit process for filtering secondary effluent dates back to the early 1950's when it was installed at the Luton Sewage Works in England. Descriptively, a microscreener consists of a rotating drum with a fine screen constituting its periphery, as illustrated in Figure 8-1 (1). Feedwater enters the drum through the open end and passes radially through the screen with concomitant deposition of solids on the inner surface of the screen. At the top of the drum, pressure jets of effluent water are directed onto the screen to remove the mat of deposited solids. The portion of the backwash stream which penetrates the screen and the dislodged solids are captured in a waste hopper, shown in Figure 8-2, and are removed through the hollow axle of the unit.

Screens employed in microscreening have extremely small openings and are made from a variety of metals and plastics. Individual manufacturers each have specific designs and sizes for the peculiar needs of any potential installation. Some examples of microfabrics available from various manufacturers are:

Opening (Microns)	No./in^2	Manufacturer
23	144,000	Crane Co., King of Prussia, Pa.
25	–	Walker Equipment Co., Chicago, Ill.
35	80,000	Crane Co., King of Prussia, Pa.
35	120,000	Zurn Industries, Inc., Erie, Pa.
40	–	Walker Equipment Co., Chicago, Ill.
60	58,500	Crane Co., King of Prussia, Pa.

The weave and shape of individual fabric wires are such that they allow the water from the backwashing jets to penetrate and detach the solids mat which forms on the inside of the screen during its passage through the feed stream. Bodien and Stenburg (2) have noted that only about one-half of the applied washwater actually penetrates the screen. The rest flows down the outer perimeter into the effluent section of the structure.

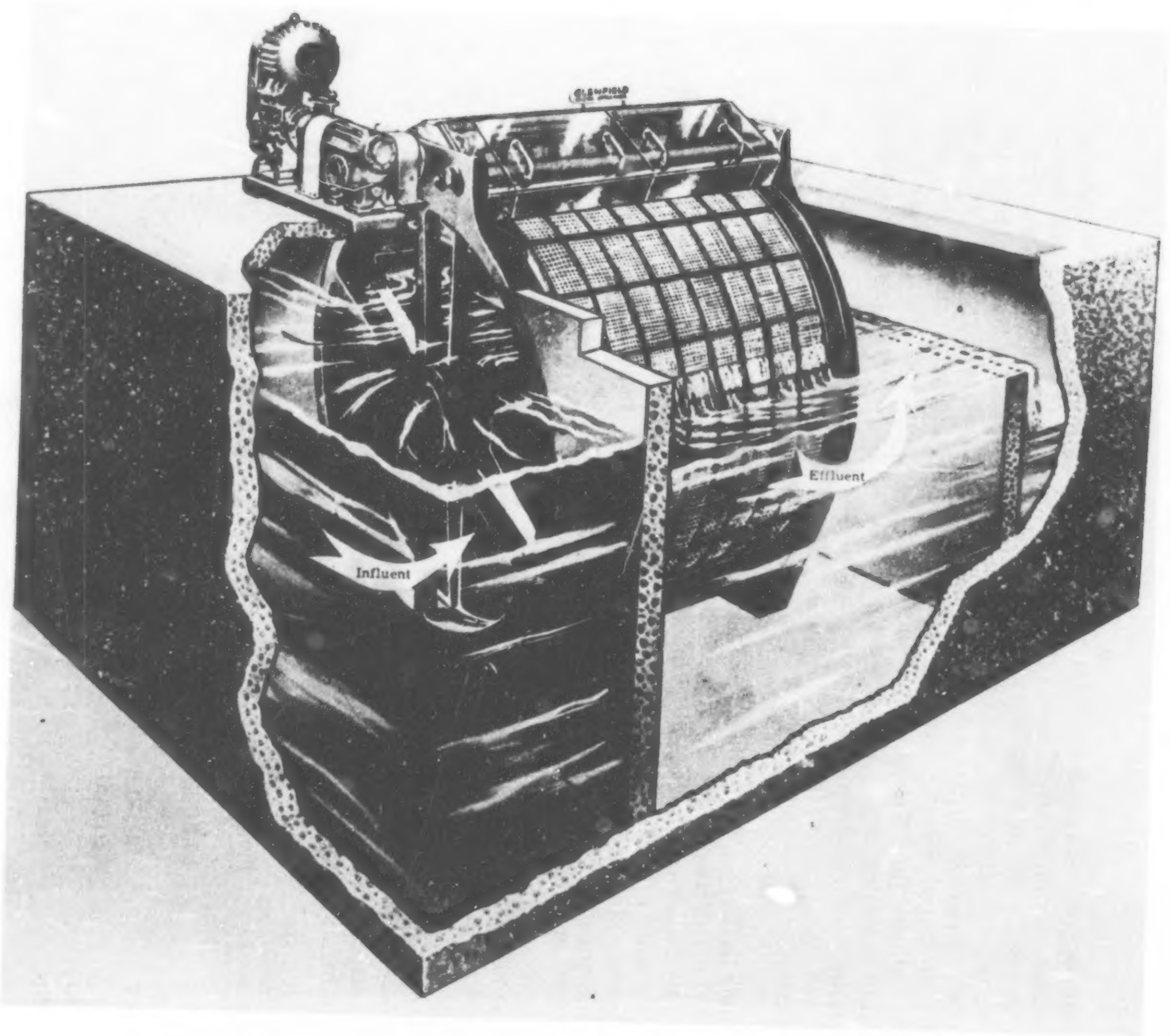

Influent
Effluent

Figure 8-1 TYPICAL MICROSCREEN UNIT
(Courtesy of Cochrane Division, Crane Co.)

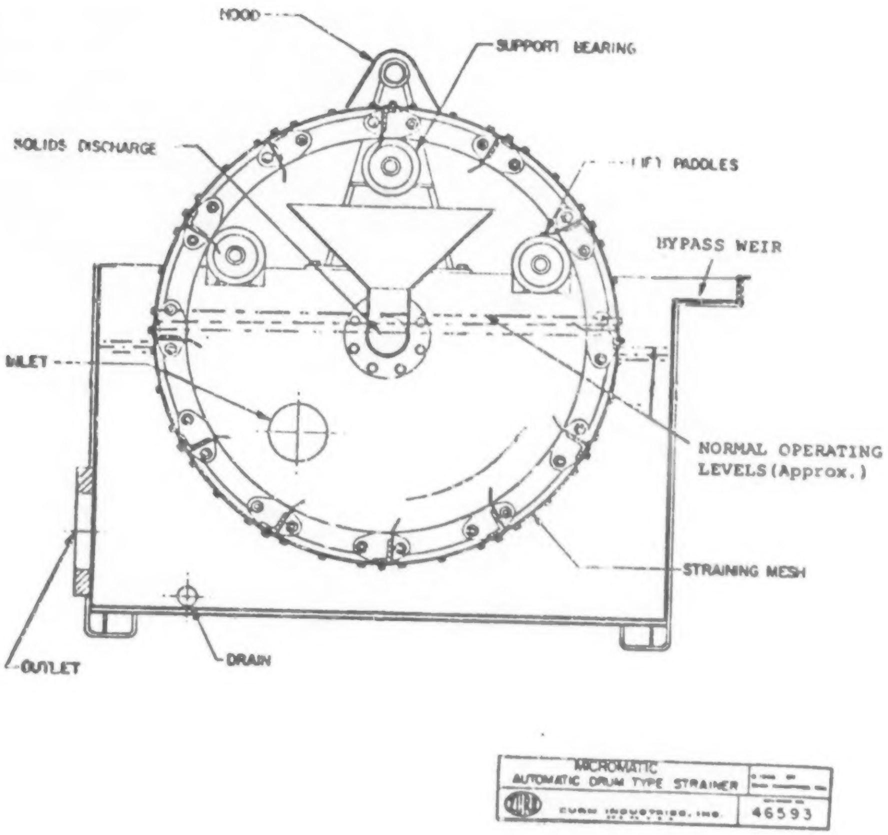

HOOD
SUPPORT BEARING
SOLIDS DISCHARGE
LIFT PADDLES
BYPASS WEIR
INLET
NORMAL OPERATING LEVELS (Approx.)
STRAINING MESH
OUTLET
DRAIN
MICROMATIC
AUTOMATIC DRUM TYPE STRAINER
ZURN INDUSTRIES, INC.
46593

Figure 8-2 TYPICAL MICROSCREEN UNIT, CROSS SECTION

Although the microscreens have small openings, they cannot account for the removal efficiency of the unit. Actually, the mat of previously-trapped solids provides the fine filtration which characterizes the unit performance. This being the case, Lynam, et al. (3), showed quantitatively that the slower the rate of drum rotation, the better the product water. Another factor which becomes important in light of this mat phenomenon is the nature of the solids applied to the microscreening process. As an example, Lynam, et al. (3), were unsuccessful in filtering the resulting chemical floc when secondary effluent was coagulated ahead of the microscreening unit.

As a section of microscreen passes through its cycle it becomes clogged rapidly as the solids mat forms. The continuous cleansing afforded by the backwashing jets at the apex of its travel must be augmented in some way to prevent the buildup of screen-clogging slimes over a period of time. An ultraviolet lamp placed in close proximity to the screen has been somewhat successful in slowing the development of these slimes. In general, however, units must be taken out of service on a regular basis to have the metal screens cleaned with a chlorine solution. In some instances, a similar cleaning with an acid solution may be occasionally required to remove iron or manganese buildups on the metal screen. In cases where oil and grease problems present themselves, a hot water and/or steam treatment can be used to remove these materials from the microscreen.

Headloss through the microscreening unit, including inlet and outlet structures, is about 12 to 18 inches (1). A 6-inch limit across the screen is usually imposed at peak flows; headlosses in excess of this value are prevented by bypass weirs, as illustrated in Figure 8-2. Headloss buildup is reduced by increasing the rate of drum rotation and by increasing the pressure and flow of the backwashing jets. These adjustments can be made manually or automatically by controls based on liquid-level sensing systems. Two-pen influent and effluent level recorders can be obtained. The effluent level pen can be used to monitor flow when an effluent weir is used. These and other control systems are available from equipment manufacturers.

The continuous backwashing jets are usually found to require from 1 to 5 percent of the total throughput volume. Higher values have been reported, up to 23 percent, during periods of secondary clarifier upsets (2). The source of backwash water is the process effluent in most cases. Pressures change during operation, as in the Chicago installation which could vary from 20 to 55 psi. The inline pressure strainer for backwash water has been found to plug from grease accumulation (3). This problem was solved by bimonthly cleaning or replacement of the strainers. Solids flushed from the screen and removed through the hopper and hollow axle are generally returned to the head of the treatment plant. Characterization of this stream has not been reported in the literature, except for one value of 700 mg/l of suspended solids (4).

Other operating parameters include the hydraulic and solids loadings on the unit. Lynam, et al. (3), found that the solids loading was the limiting factor in microscreening activated sludge secondary effluent. Maximum capacity was found to be 0.88 lb/day/sq ft at an hydraulic loading of 6.6 gpm/sq ft. Excessively high solids during upset periods can reduce the hydraulic loading of the unit drastically from design levels. One of the advantages of using microscreens is their low head requirement. It is feasible, therefore, and advantageous to conduct secondary effluent without pumping to a tertiary microscreening installation in order to minimize the shear forces imparted to the biological floc. Chlorination immediately ahead of microscreening units should be avoided to protect the screens.

The primary material of construction for microscreeners is carbon steel. The numerous appurtenances, support members and fittings are made from a variety of materials. Smaller units can be factory assembled in steel tanks (Figure 8-3), while larger units (Figure 8-4) are generally placed in concrete substructures, constructed according to the manufacturers specifications (Figure 8-5) (1). One manufacturer, Crane Co., King of Prussia, Pa., indicates the overall dimensions of four standard microscreening installations in Figure 8-6. For those same units the necessary drive and backwashing pump horsepowers are indicated in Table 8-1, along with the approximate capacities of these drum sizes. In larger installations, more than one unit per tank may be employed.

The design of microscreening units has been based on an equation derived by Boucher. This equation is:

$$H = \frac{mQCf}{A} (e)^{nIQ/S}$$

where:

H = headloss across the strainer

Q = constant total rate of flow through unit

Cf = initial headloss through strainer

A = effective submerged area

I = filterability index of influent feed

S = speed of strainer (rate of area exposure)

When H is expressed in inches, Q in gpm, Cf in feet, A in square feet and S in square feet/minute, m is 0.0267 and n is 0.1337. The key to the successful use of this equation is an accurate determination of the filterability index, I, which is defined as the volume of water obtained, per unit headloss, when passed through a unit area of standard filter. This can be determined accurately by a laboratory procedure (5) or roughly by a field testing apparatus, as shown in Figure 8-7 (1). Critical values of headloss, drum speed and filtrability index can then be employed in conjunction with the aforementioned equation to determine design requirements. One investigator has proposed a modification to the design equation for improved accuracy (6).

Microscreening devices have generally been applied as tertiary screens for the filtration of secondary effluents from trickling filter and activated sludge treatment. Some attempts have been made to utilize a microscreening unit in place of a secondary clarifier for trickling filters in Great Britain, but these have been only partially successful because of the strict dependence of the unit on the performance of the trickling filter, i.e., good trickling filter performance yields good microscreening removals, while poor performance plays havoc with the microscreener (4). Tertiary applications have been more frequent in Great Britain, but interest and testing in the United States have been increasing with the advent of more stringent requirements on treatment plant effluents. Published results have been generally limited to a single manufacturer's product, but a number of other equipment firms have recently entered the market with similar devices.

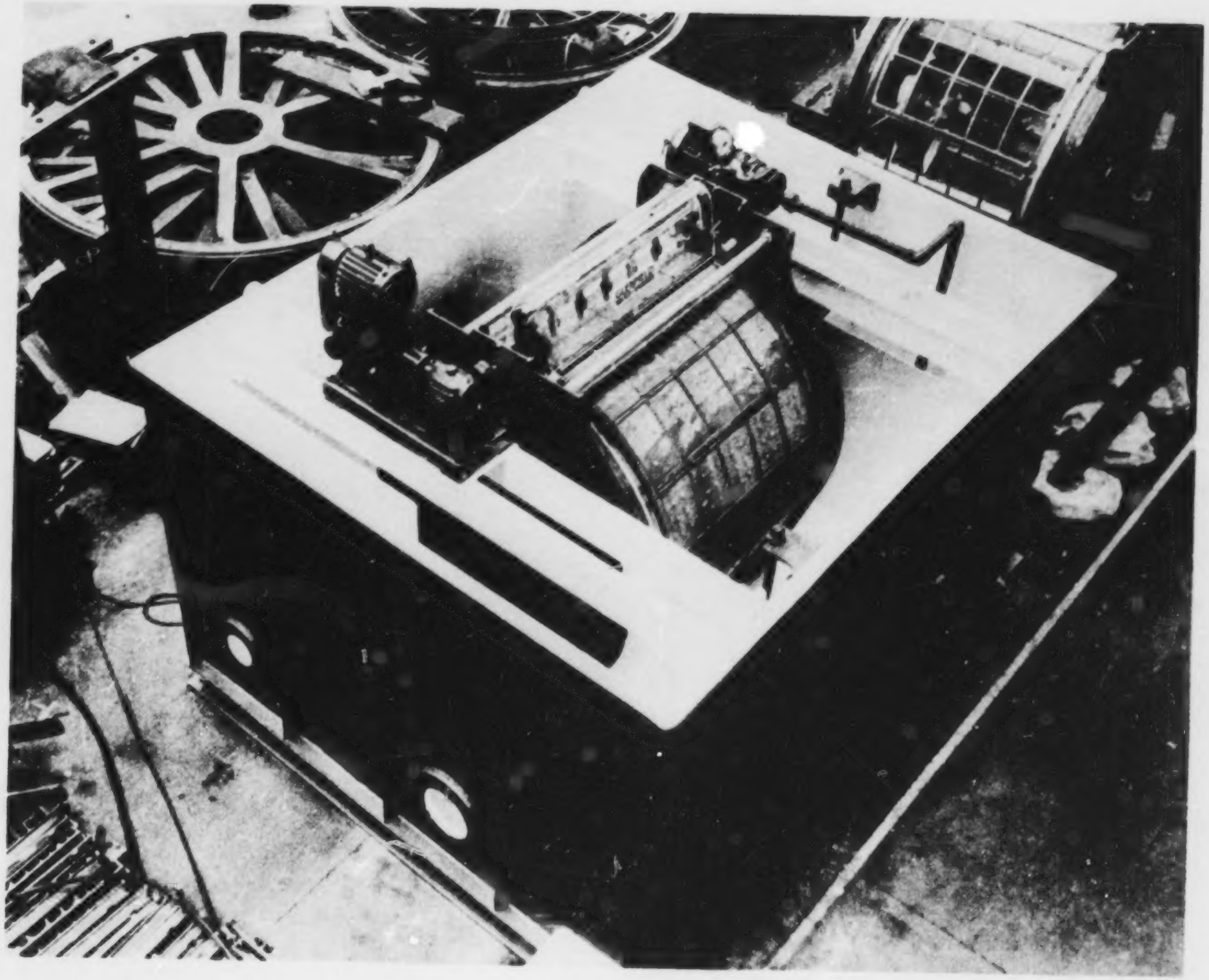


Figure 8-3 MICROSCREEN INSTALLED IN STEEL TANK AT FACTORY
(Courtesy of Crane Co.)

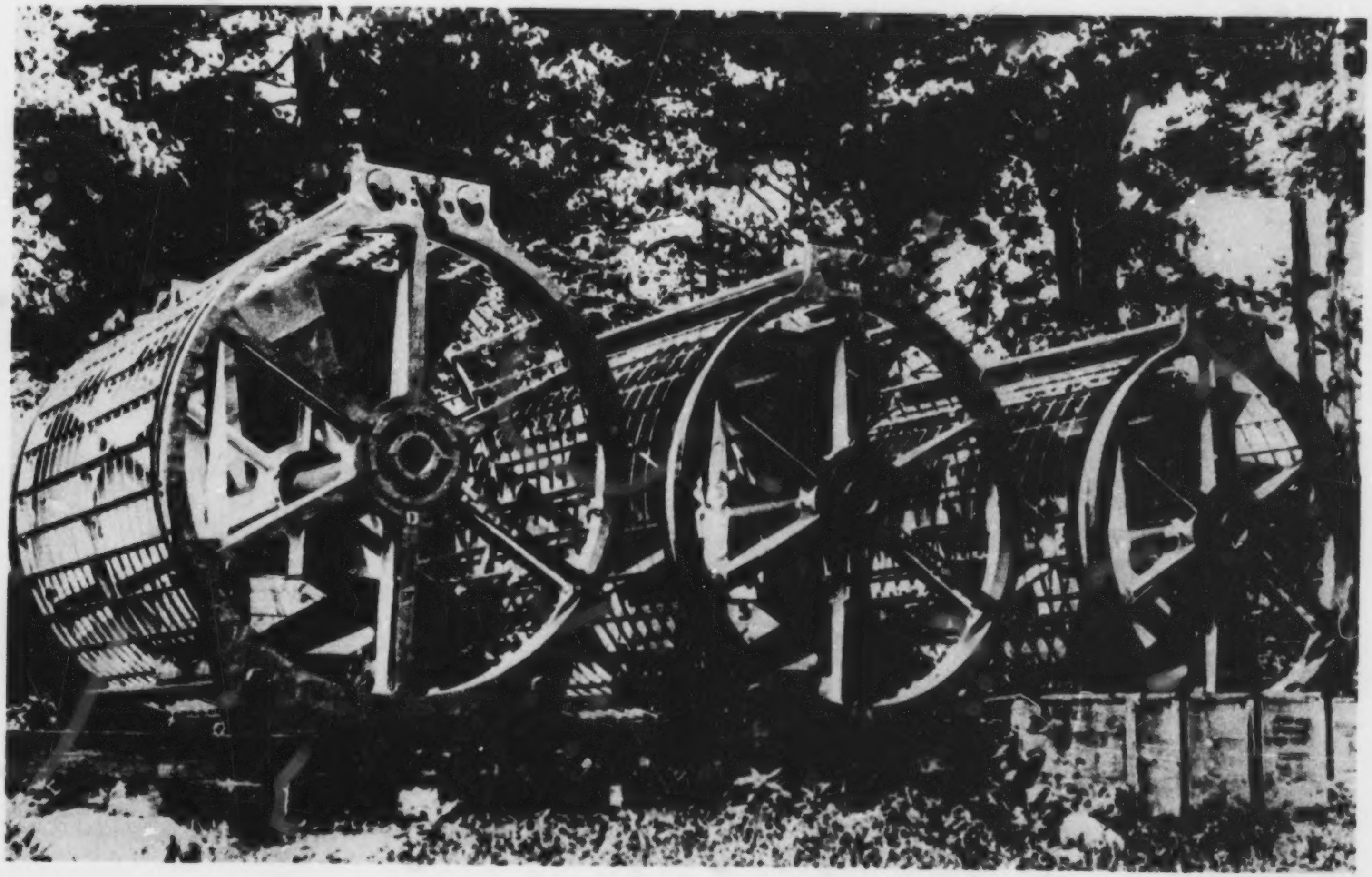


Figure 8-4
LARGE MICROSCREEN DRUMS PRIOR TO
INSTALLATION AT CHICAGO, ILLINOIS

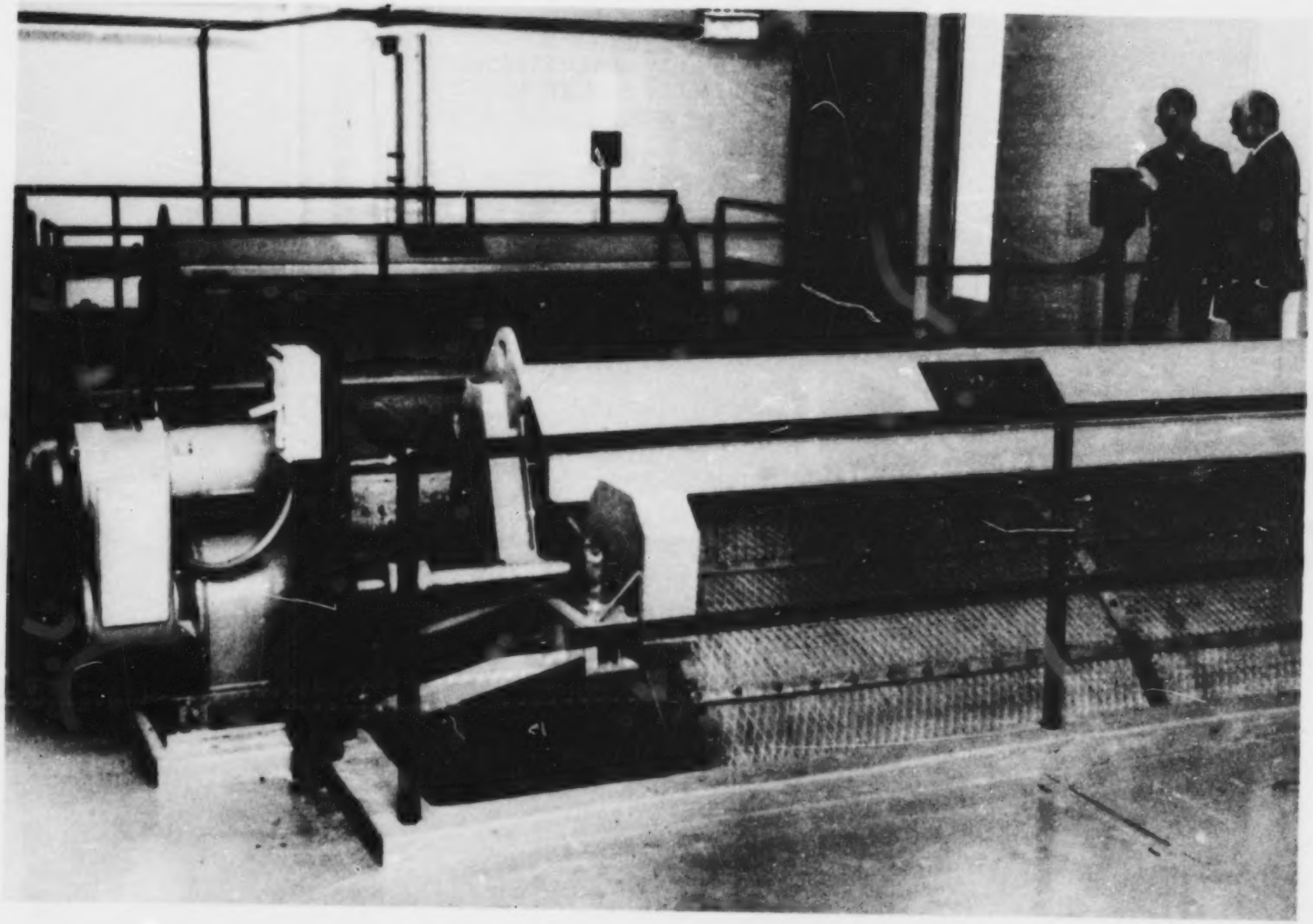
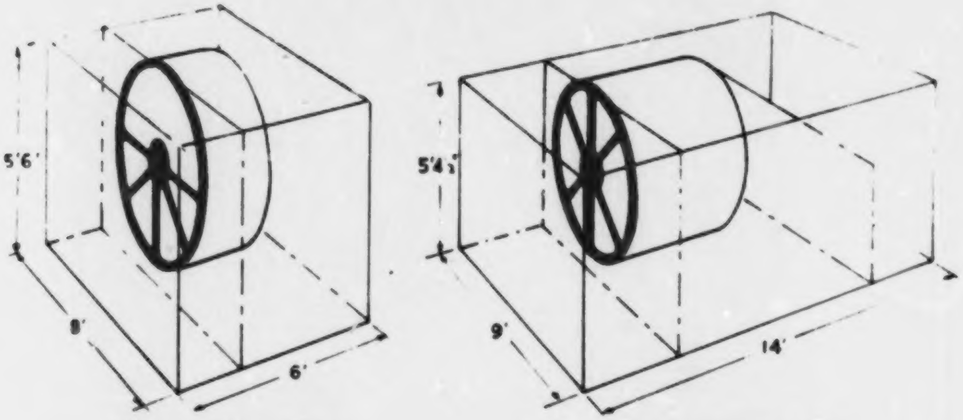
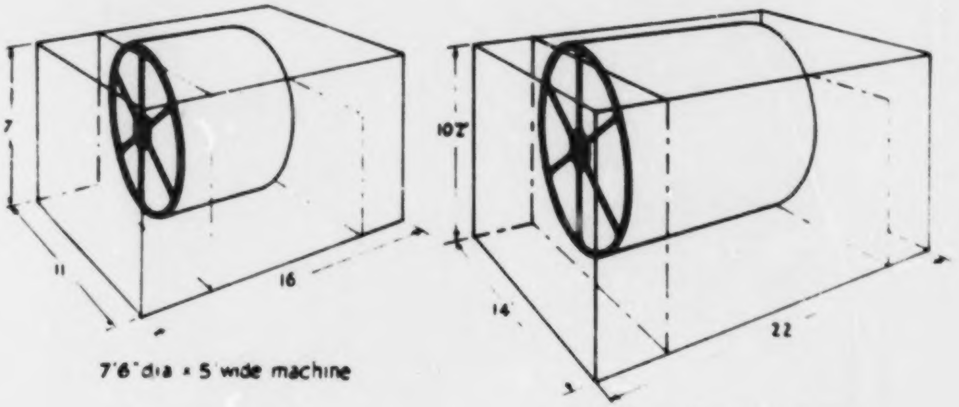


Figure 8-5 LARGE MICROSCREENS IN OPERATION
(Photo Courtesy Zurn Industries, Inc.)


5'6"
8'
6'
5'4½"
9'
14'

5' dia × 1' wide machine

5' dia. × 3' wide machine


7
11
16
10'2"
14'
22

7'6" dia × 5' wide machine

10' dia × 10' wide machine

Figure 8-6
OVERALL DIMENSIONS FOR FOUR SIZES OF MICROSCREENERS

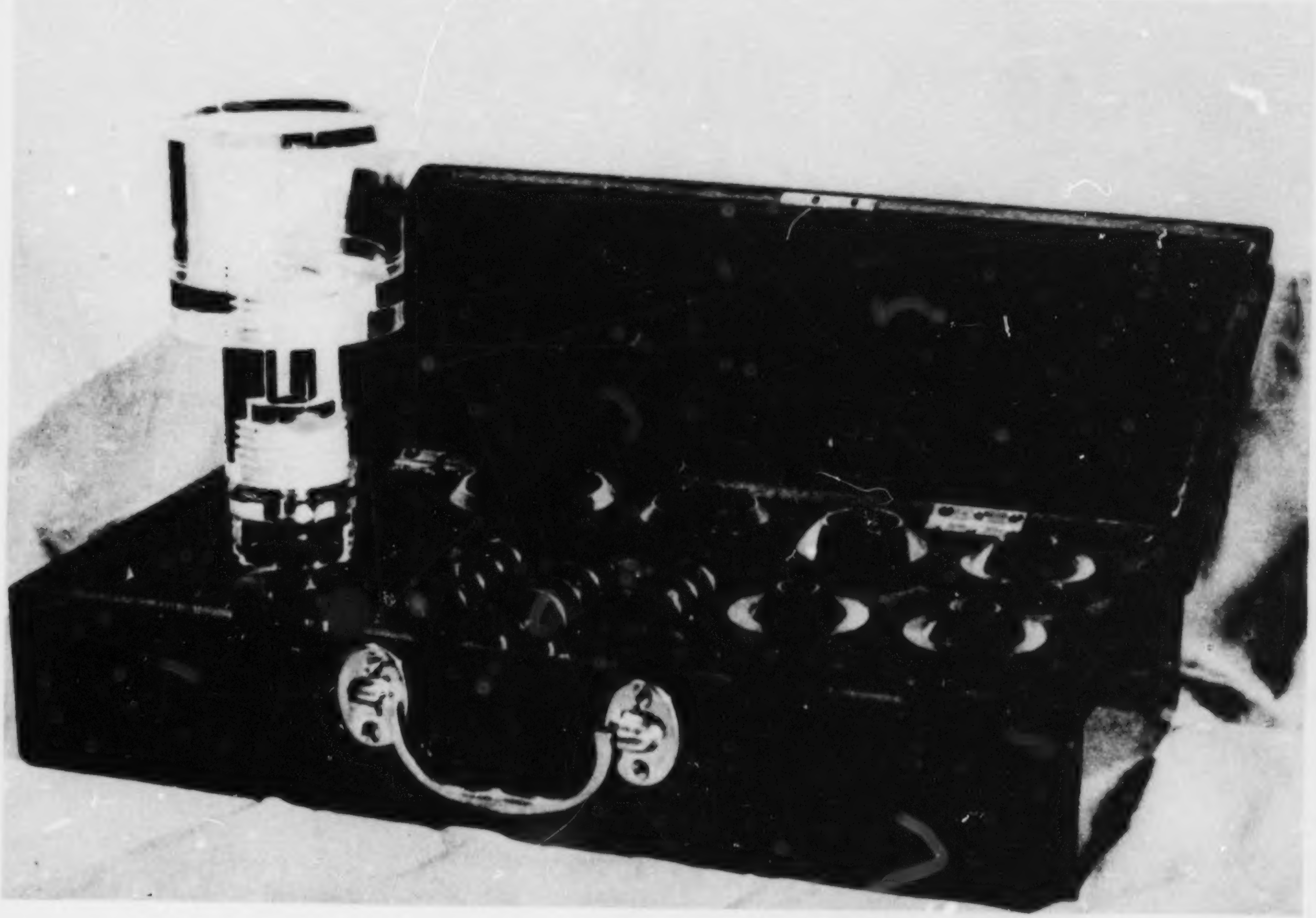


Figure 8-7
LABORATORY FOR DETERMINATION OF FILTERABILITY INDEX
(Courtesy Crane Co.)

Table 8-1

MICROSCREENER SIZES, MOTORS AND CAPACITIES

Drive Sizes (ft) Diam.	Drive Sizes (ft) Width	Motors (BHP) Drive	Motors (BHP) Wash Pump	Approx. Ranges of Capacity (MGD)
5.0	1.0	0.50	1.0	0.1 to 0.5
5.0	3.0	0.75	3.0	0.3 to 1.5
7.5	5.0	2.00	5.0	0.8 to 4.0
10.0	10.0	5.00	7.5	3.0 to 10.0

A number of installations and some published results are shown in Table 8-2. Suspended solids are the variable of primary interest as the BOD removals are merely a function of the solids composition.

From the table it can be seen that the finer screen produced somewhat better solids removal averages, but direct quantitative comparison is not possible in this case.

8.3 Other Screens

Rotary screens, similar in principle and appearance to microscreeners, are available for more gross solids removal. Also, vibratory self-cleaning screens are being used for similar purposes. Their major use is generally in industrial waste treatment to remove the coarser solids prior to disposal or further treatment. Recently, these self-cleaning rotary and vibratory screens have been utilized for treatment of combined sewer overflows with some success. Their advantage in such applications is owed to their compact size and their ability to operate on a start and stop mode. Suspended solids removals for these systems have generally ranged from 20 to 40 percent (7), (8).

8.4 Diatomaceous Earth Filters

Diatomaceous earth (DE) filters have been applied for the clarification of secondary effluents at pilot scale. No full-scale installations have been characterized in the literature. DE filtration utilizes a thin layer of precoat formed around a porous septum to strain out the suspended solids in the feedwater which passes through the filter cake and septum. The driving force can be imposed by vacuum from the product side or pressure from the feed side. As filtration proceeds, headloss through the cake increases due to solids deposition until a maximum is reached. The cake and associated solids are then removed by flow reversal and the process is repeated. In the cases where secondary effluents have been treated by this process, a considerable amount of diatomaceous earth (body feed) has been required for continuous feeding with the influent in order to prevent rapid buildup of headlosses and subsequent uneconomically short filter runs. Generally, the DE filtration process is capable of removing suspended solids, but not colloidal matter. A schematic drawing of a DE system during operation is shown in Figure 8-8.

Table 8-2

TERTIARY TREATMENT BY MICROSCREENERS

Location	Screen Size (Microns)	Plant Size (MGD)	S.S. (Rem. %)	BOD (Rem. %)	Backwash (%)	Manufacturer
Luton, England	35	3.6	55	30	3.0	Crane
Bracknell, England	35	7.2	66	32	N.A.	Crane
Harpendon, England	35	0.3	80	N.A.	N.A.	Crane
Brampton, Ontario	23	0.1	57	54	N.A.	Crane
Chicago, Illinois	23	2.0	71	74	3.0	Crane
Lebanon, Ohio	23	Pilot	89	81	5.3	Crane
	35	Pilot	73	61	5.0	Crane
Miami, Florida (S. Gulf Utilities)	N.A.	N.A.	N.A.	N.A.	N.A.	Zurn
Islip, New York	N.A.	0.165	99	96	N.A.	Zurn
Murfreesboro, Tennessee	N.A.	N.A.	N.A.	N.A.	N.A.	Zurn
Howell Twp., New Jersey	N.A.	N.A.	N.A.	N.A.	N.A.	Walker
Columbia, South Carolina	N.A.	N.A.	N.A.	N.A.	N.A.	Walker
Macomb, Illinois	N.A.	N.A.	N.A.	N.A.	N.A.	Walker

N.A. = Not available

A wide variety of diatomaceous earth (diatomite) grades are available for use. As might be expected, the coarser grades have greater permeability and solids-holding capacities than do the finer grades which will generally produce a better effluent. Fly-ash has been used as body feed, also. Some grades of diatomite are pretreated to change their characteristics for improved performance. A number of vessel configurations are also available,as illustrated in Figures 8-9 through 8-14.

Design criteria for diatomite filters have been discussed by Bell (10). The filtration cycle can be divided into two phases,run time and down time. Down time includes the periods when the dirty cake is dislodged from the septum and removed from the filter and when the new precoat is formed. Most of the difficulties involved in these operations are dependent upon the equipment design of the individual manufacturer. Run time commences when the feed is introduced to the filter and ends when a limiting headloss is reached or an effluent quality degradation occurs. The single most important problem with secondary effluent filtration by DE filters is the amount of body feed required during the filtration or run time. A ratio of 5 to 6 mg/l of body feed per JTU of influent turbidity was required at San Antonio and the possible need for a higher ratio was suggested (11). Another pilot study used a variety of ratios and filtration rates and used both pressure and vacuum systems for secondary effluent filtration (12). Results of this study are shown in Tables 8-3 and 8-4. Both studies indicate that a precoat of about 0.1 lb/sq ft of filter area, high ratios of body feed to influent turbidity and a filtration rate of approximately 0.5 gpm/sq ft of filter area should be used.

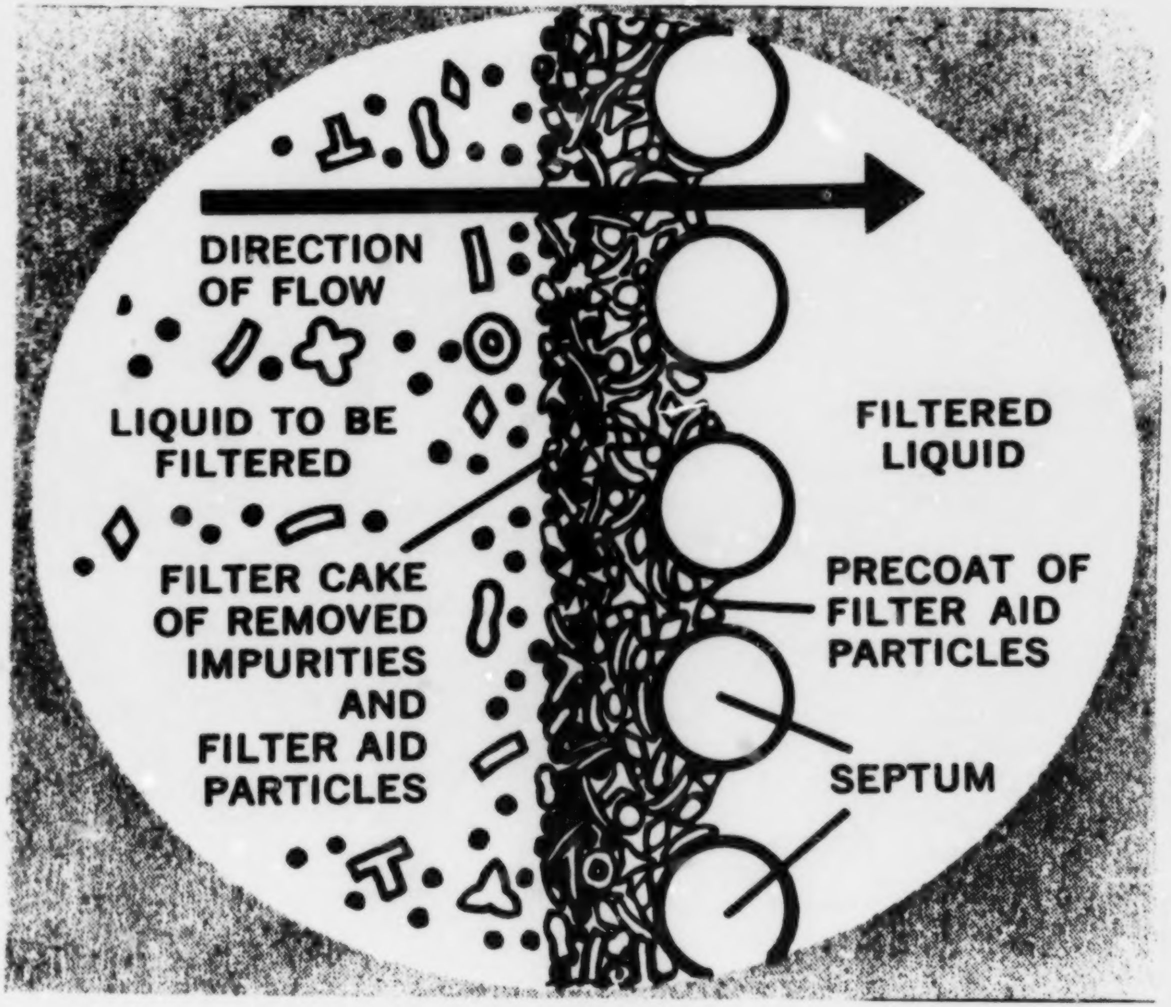

DIRECTION OF FLOW
LIQUID TO BE FILTERED
FILTER CAKE OF REMOVED IMPURITIES AND FILTER AID PARTICLES
FILTERED LIQUID
PRECOAT OF FILTER AID PARTICLES
SEPTUM

Figure 8-8

DIATOMITE FILTRATION SYSTEM DETAIL

(Courtesy of Johns-Manville)

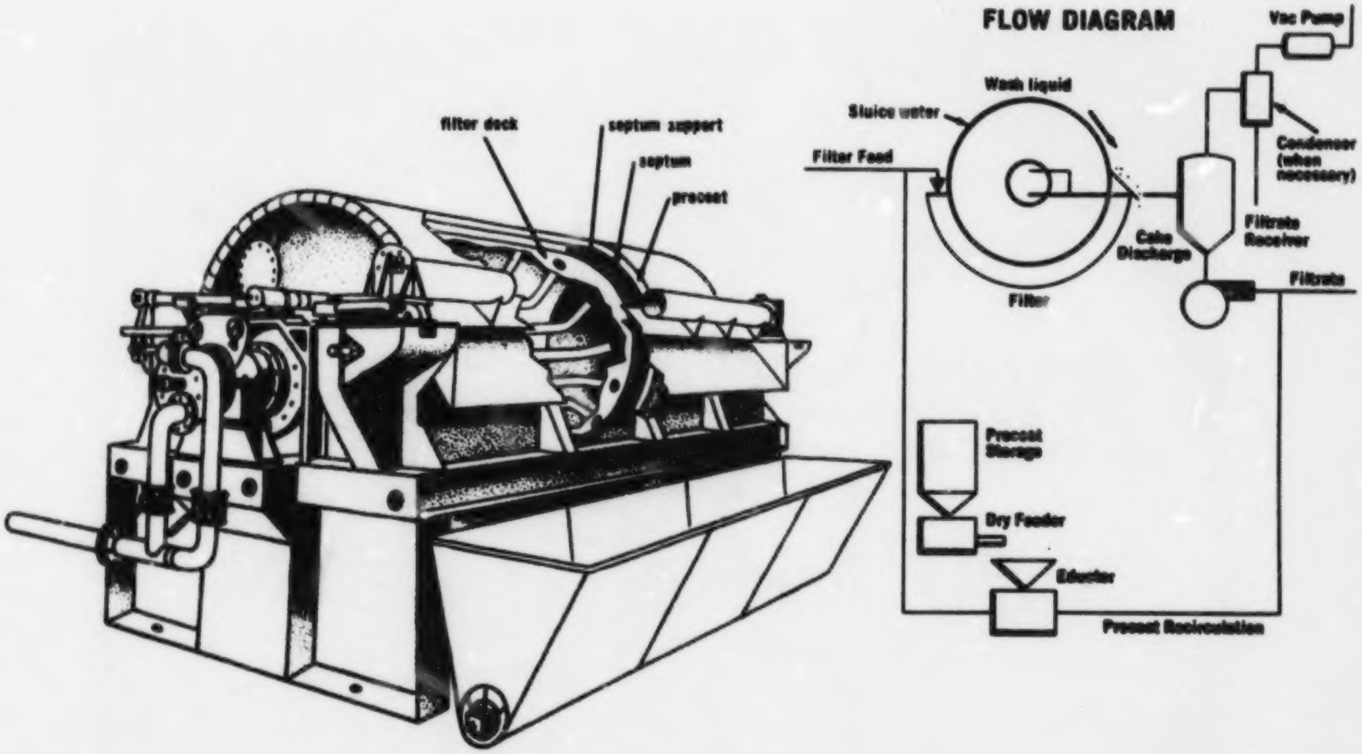

filter deck
septum support
septum
precoat
FLOW DIAGRAM
Vac Pump
Wash liquid
Sluice water
Filter Feed
Condenser (when necessary)
Cake Discharge
Filtrate Receiver
Filtrate
Filter
Precoat Storage
Dry Feeder
Eductor
Precoat Recirculation

Figure 8-9 ROTARY VACUUM PRECOAT FILTER
(Courtesy of Johns-Manville)

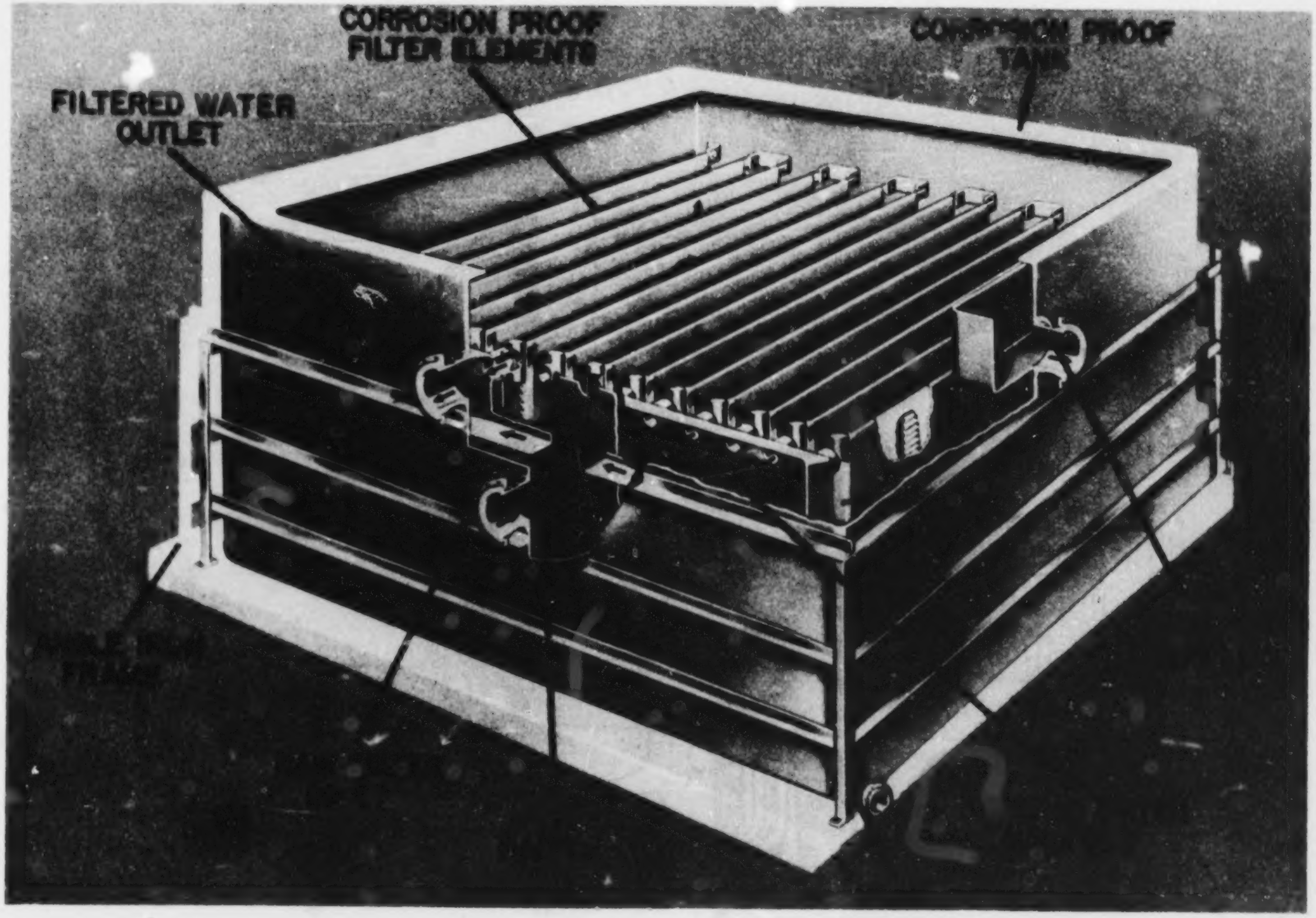

CORROSION PROOF FILTER ELEMENTS
CORROSION PROOF TANK
FILTERED WATER OUTLET

Figure 8-10 VERTICAL LEAF VACUUM FILTER
(Courtesy of Johns-Manville)

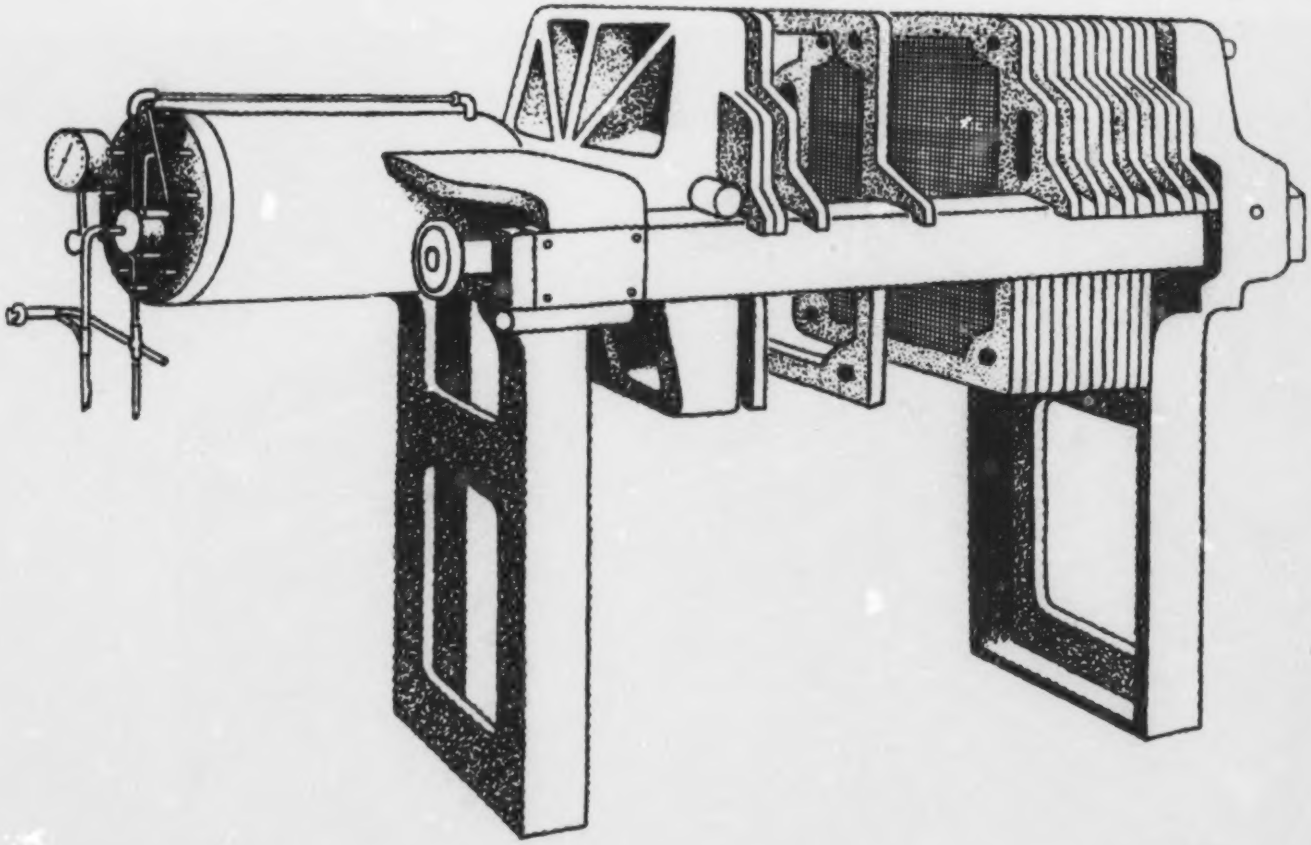


Figure 8-11 PLATE AND FRAME PRESSURE FILTER
(Courtesy of Johns-Manville)

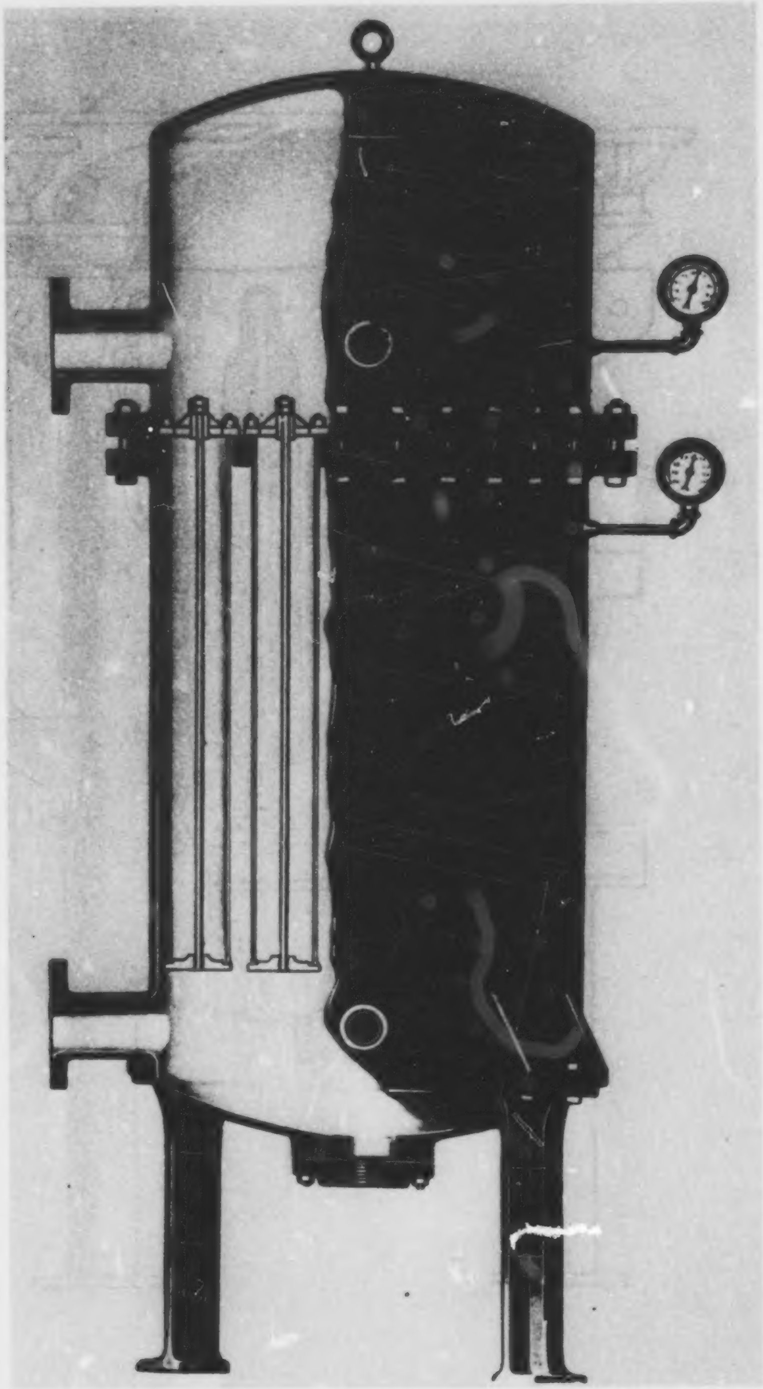


Figure 8-12 CYLINDRICAL ELEMENT VERTICAL FILTER
(Courtesy of Johns-Manville)

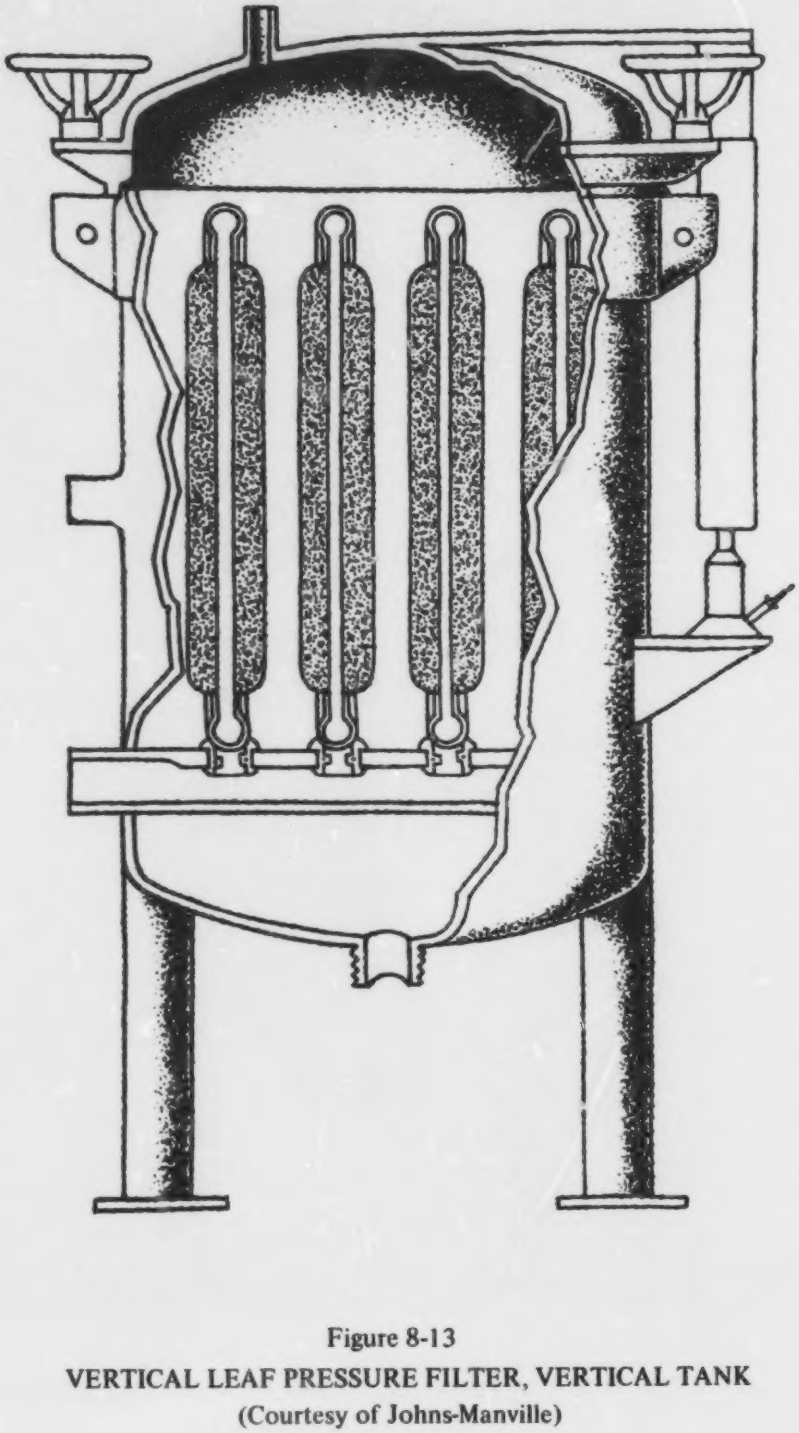


Figure 8-13
VERTICAL LEAF PRESSURE FILTER, VERTICAL TANK
(Courtesy of Johns-Manville)

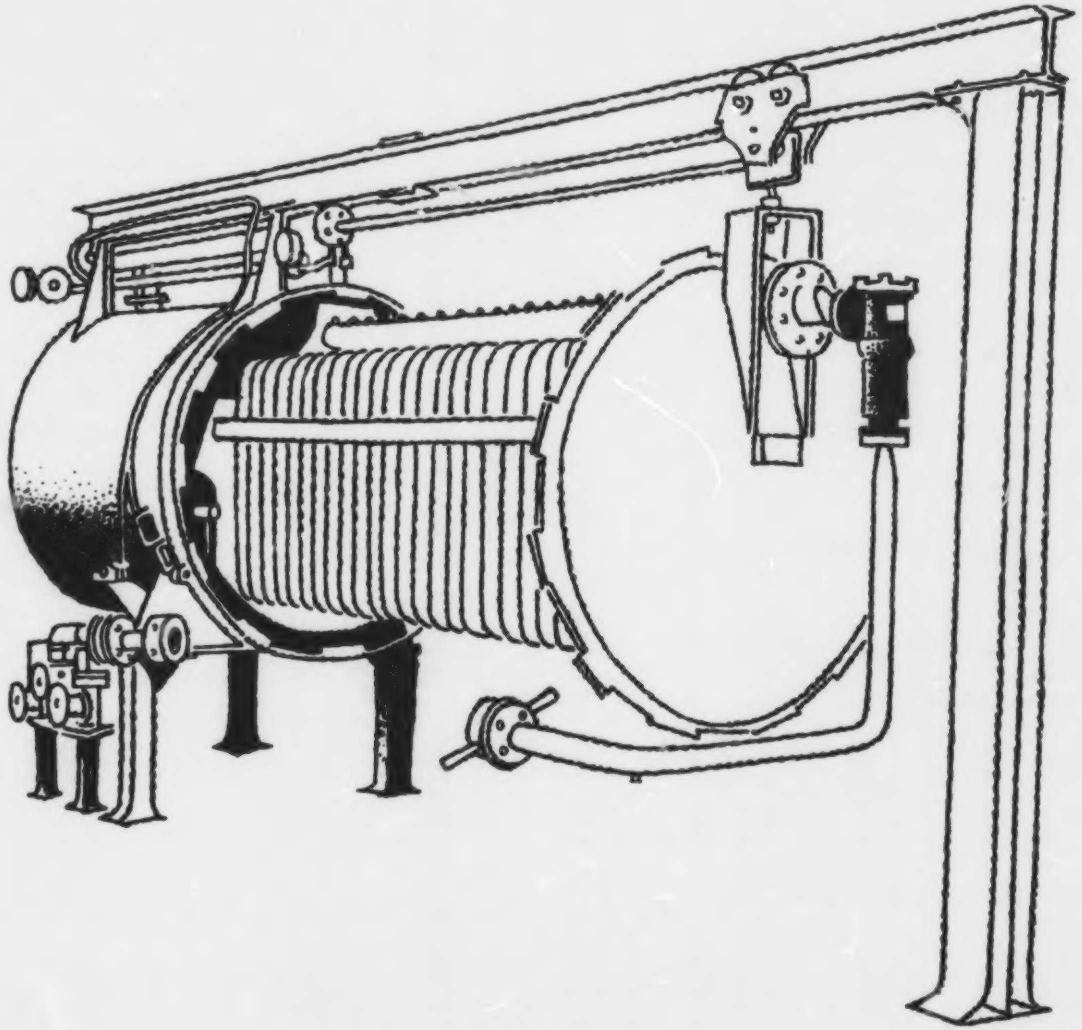


Figure 8-14
VERTICAL LEAF PRESSURE FILTER, HORIZONTAL TANK
(Courtesy of Johns-Manville)

Table 8-3

VACUUM DIATOMACEOUS EARTH FILTRATION OF SECONDARY EFFLUENT

Flow Rate (gpm/ft^2)	Filter Aid	Body-feed Conc. (mg/1)	No. of Runs	Run-length (hrs)	Turbidities (JTU)* Feed	Turbidities (JTU)* Product
0.53	Celite 545**	42	5	19.5	5.5	0.8
0.75		33	3	10.7	5.2	0.8
1.0		19	1	5.4	4.4	0.4
0.51	Celite 503**	36	2	8.0	10.5	3.3
0.75		29	1	3.0	9.2	0.7
1.0		21	1	1.2	8.0	0.75
0.54	Hyflo	35	6	12.7	4.6	0.85
0.75	Super-Cel **	21	2	4.4	5.8	0.8

Precoat – 0.1 lb/ft^2
Headloss at end of run – 18 in. Hg
*Jackson Turbidity Units
**Product of Johns-Manville, New York, N.Y. 10016

Table 8-4

PRESSURE DIATOMACEOUS EARTH FILTRATION OF SECONDARY EFFLUENT

Flow Rate (gpm/ft^2)	Filter Aid	Body-feed Conc. (mg/1)	No. of Runs	Run-length (hrs)	Turbidities (JTU)* Feed	Turbidities (JTU)* Product
0.50	Celite 545**	50	1	50.0	8.2	3.1
0.75		19	2	24.2	5.7	2.5
0.81		42	2	28.4	8.3	3.9
1.0		20	2	7.3	6.4	2.1
1.0		45	3	31	7.5	3.0
0.76	Celite 503**	18	1	14.5	7.0	4.0
0.78	Hyflo	21	1	22.3	8.1	4.9
1.2	Super-Cel **	29	1	9.7	6.5	3.8

Precoat – 0.1 lb/ft^2
Head loss at end of run – 35 psi

*Jackson Turbidity Units
**Product of Johns-Manville, New York, N.Y. 10016

8.5 Ultrafiltration

All membrane processes share certain operating and design aspects which can be discussed before ultrafiltration is set into context (13). In order to separate dissolved materials from water, a relatively "fine" membrane with relatively high pressures and low flow rates is required. When particles in the colloidal and suspended range must be separated from water, the membrane can be somewhat more "coarse" with correspondingly lower pressures; however, the flux, or flow through the membrane, may be substantially the same as for salt separation. A problem common to both the salt and solids removal applications is that of maintaining membrane flux in the face of particulate or precipitate fouling of the membrane surface. The flux tends to decline logarithmically with time.

The salt and solids removal applications have in common those aspects just discussed. In every other respect they are different. It is therefore appropriate that two different process names be applied to them. Salt removal is called reverse osmosis or hyperfiltration, while solids removal is called ultrafiltration. Hyperfiltration has fluxes from 5 to 50 gpd/sq ft of membrane area at pressures of 500 to 1500 psig. Performance usually includes 90-99% rejection of total dissolved solids and 100% of suspended solids. Ultrafiltration may exhibit 5-50 gpd/sq ft of flux, but requires pressures of only 10-100 psig and removes virtually 100% of the suspended material. Little operating information for ultrafiltration is available, and it will be considered only briefly.

An ultrafiltration process flow diagram is shown in Figure 8-15. The two most important elements of ultrafiltration are the membrane and the geometrical configuration into which it is set. The exact operating and performance data depend upon the nature of the waste. Given the current state of membrane-manufacturing technology, almost any set of desired performance characteristics can be produced by a membrane. Specifications include membrane flux and flux decline characteristics. Rather than discuss membrane capabilities and formulas in detail, it is sufficient to note that a few of the leading manufacturers of ultrafiltration membranes are Amicon Corp., Abcor, Inc. and Dorr-Oliver, Inc. Catalogues offering a wide variety of membranes are available.

The geometrical configuration of the module containing the membrane is just as important as the membrane itself. The single greatest problem of ultrafiltration or any other membrane process is the fouling which causes membrane flux to decline. Such fouling is caused by slimes, precipitates and organic and microbial deposits. The membrane can be cleaned routinely with chemical or enzyme solutions, but prevention of this fouling is an important design problem. The precise nature of the fouling depends on the nature of the feedwater. Prevention of fouling may be accomplished by achievement of proper hydrodynamic conditions within the module. The feedwater velocity past the membrane surface serves to continually scour that surface to prevent fouling.

Another aspect of module design concerns the magnitude of membrane surface area which can be incorporated into a module. Because of the low membrane fluxes it is imperative to design the module so as to maximize membrane surface area. One configuration adopted solely for ultrafiltration is the storage battery configuration, as shown in Figure 8-16. The membrane is cast on both external faces of a hollow plate, and many of these plates are arranged in a parallel array. The edges of these plates face the incoming stream of solids, and thereby function as a coarse screen which can be backwashed by reversing the approach

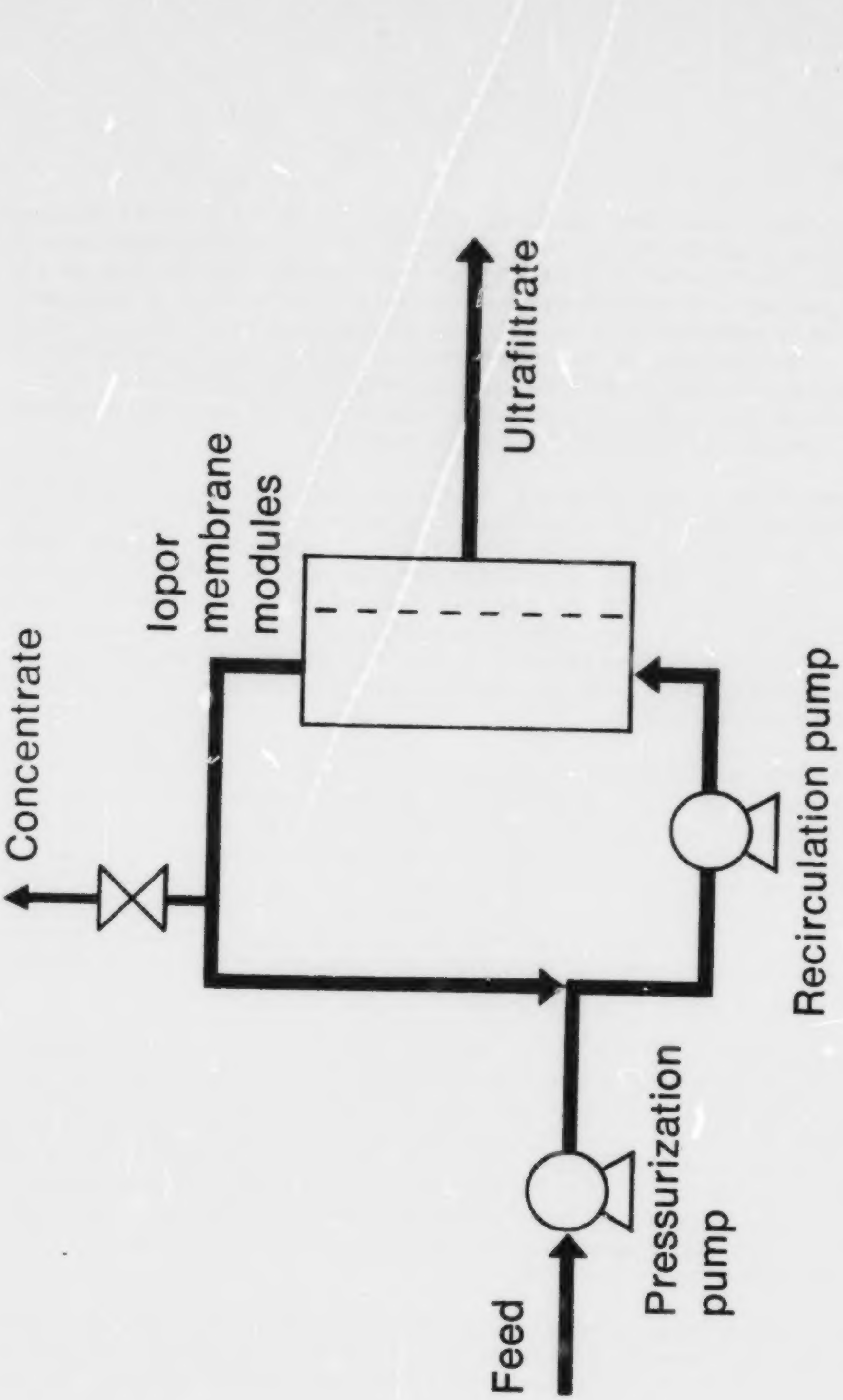

Feed
Pressurization pump
Recirculation pump
Concentrate
Iopor membrane modules
Ultrafiltrate

Figure 8-15 ULTRAFILTRATION FLOW DIAGRAM
(Courtesy Dorr-Oliver, Stamford, Conn.)

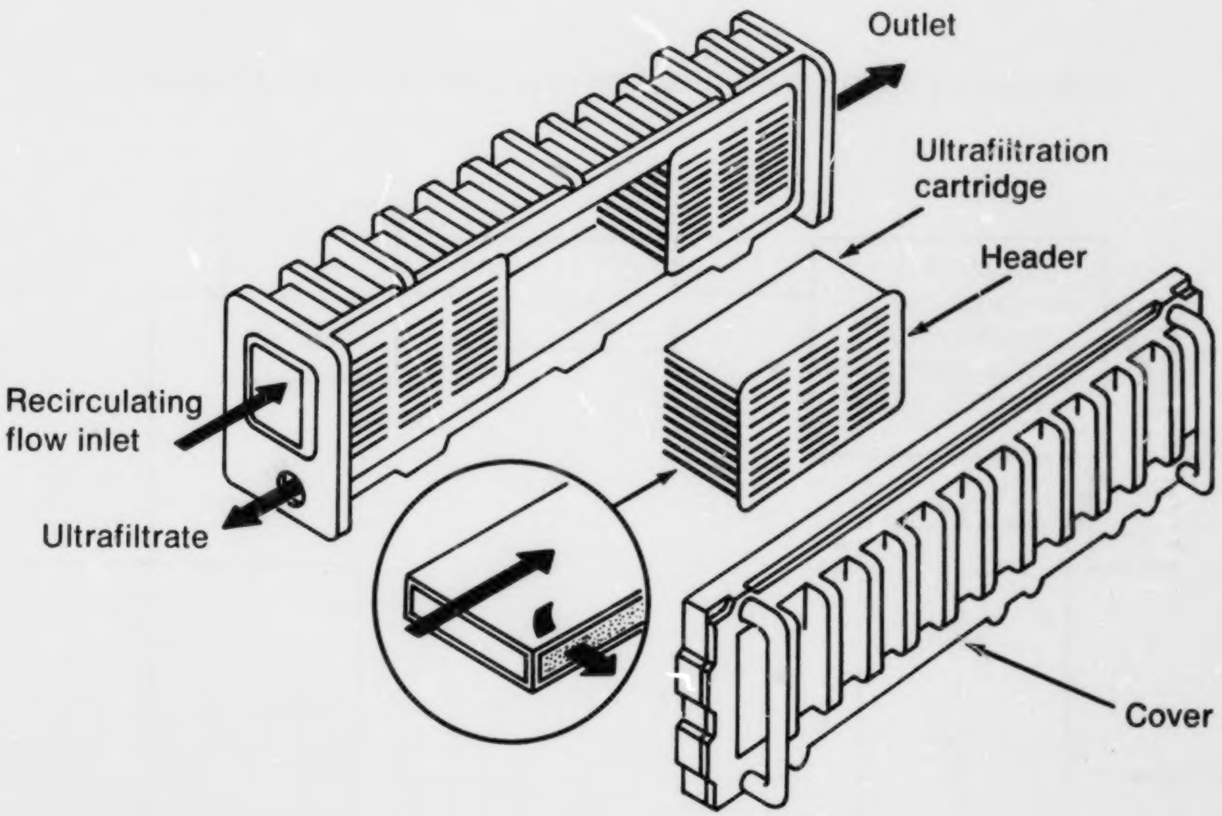

Outlet
Ultrafiltration cartridge
Header
Recirculating flow inlet
Ultrafiltrate
Cover

Figure 8-16 "STORAGE BATTERY" MEMBRANE MODULES
(Courtesy of Dorr-Oliver).

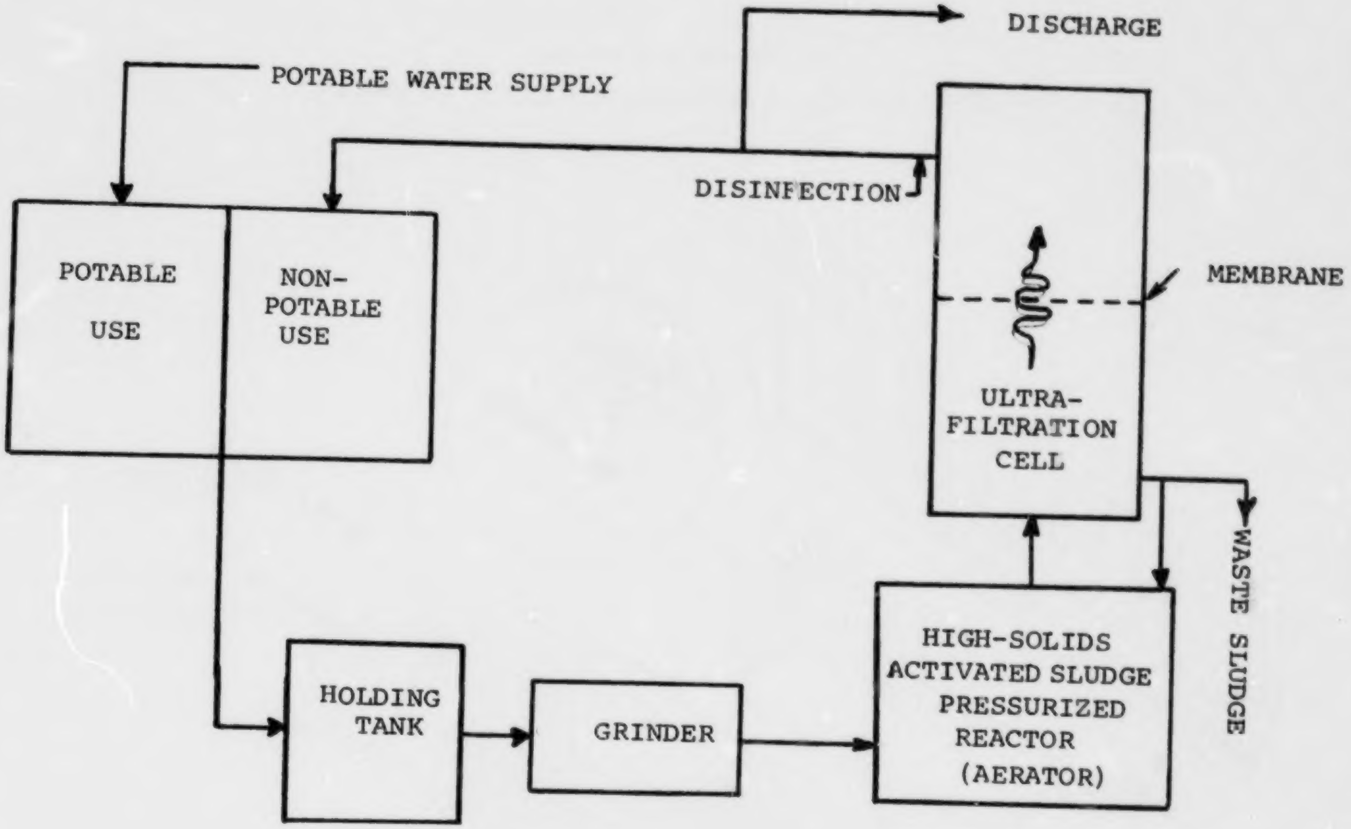

POTABLE WATER SUPPLY
DISCHARGE
DISINFECTION
MEMBRANE
POTABLE USE
NON-POTABLE USE
ULTRA-FILTRATION CELL
WASTE SLUDGE
HOLDING TANK
GRINDER
HIGH-SOLIDS ACTIVATED SLUDGE PRESSURIZED REACTOR (AERATOR)

Figure 8•17
SCHEMATIC FLOW DIAGRAM OF THE PIKES PEAK TREATMENT & REUSE SYSTEM

direction of the feed. The influent passes through the membrane-plates and is collected by a manifold connecting the interiors of the various hollow plates. This design is advantageous for three reasons: (a) it is compact and incorporates a large membrane area into the module; (b) the edges of the plates function as an easily backwashed coarse filter; and (c) the flow is parallel to the membrane surface which induces scouring.

Ultrafiltration is currently competitive with other solids removal processes only in specialized situations. On account of its compactness, an ultrafiltration module can easily replace a filter or settler in a package treatment plant where space is at a premium. A package plant on top of Pikes Peak (14) treats 21,000 gpd by a high-solids activated sludge and ultrafiltration process. The concentrate from the ultrafiltration step is recycled to the aerator as shown in Figure 8-17. Table 8-5 shows the performance data collected during a brief period in 1970 after the plant was installed. The plant had not yet reached peak performance in some respects, but its chief merits are evident.

Table 8-5

SUMMARY OF PERFORMANCE OF THE
DORR-OLIVER ACTIVATED SLUDGE-ULTRAFILTRATION PLANT OPERATIONS AT PIKES PEAK

AUGUST-SEPTEMBER, 1970

Parameter	Influent (mg/l)	Effluent (mg/l)	Removal (%)
BOD	285	1	99
COD	547	32	94
TOC	136	6.6	95
Turbidity (JTU)	47	0.33	–
Color (Units)	320	40	–
TSS	129	0	100
MLSS	3954	–	–
Coliform (Per 100 ml)	–	0	100
PO_4 - P	9.1	11.1	–
pH	7.9	5.9	–
Threshhold Odor Number	–	6	–

Pressure ~ 50 psig
Average Flux – 21,000 GPD

8.6 References

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2. Bodien, D.G., and Stenburg, R.L., "Microscreening Effectively Polishes Activated Sludge Effluent", Water & Wastes Engrg., 3, 74 (Sept. 1966).

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5. Boucher, P.L., "A New Measure of the Filterability of Fluids with Application to Water Engineering", ICE Jour. (Brit.), 24, 4, 415 (1947).

6. Mixon, F.O., "Filterability Index and Microscreener Design", Jour. WPCF, 42, 1944 (Nov. 1970).

7. Mason, D.G., "The Use of Screening/Dissolved-Air Flotation for Treating Combined Sewer Overflow", Paper presented at Seminar on Storm & Combined Sewer Pollution Problems (Nov. 1969).

8. Cornell, Howland, Hayes and Merryfield, "Rotary Vibratory Fine Screening of Combined Sewer Overflows", U.S.D.I., FWQA Report No. 11023 EDD (Mar. 1970).

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11. Wells, W.N., and Davis, D.W., "Filtration of Activated Sludge Plant Effluent", Public Works, 98, 4, 94 (Apr. 1967).

12. "Summary Report - Advanced Waste Treatment Research Program (1964-1967)", U.S.D.I., FWPCA Publication WP-20-AWTR-19 (1968).

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CHAPTER 9

DEEP BED FILTRATION

9.1 General

With the exception of gravity sedimentation, deep-bed filtration is the most widely used unit process for liquid-solids separation. Until recently its use was generally confined to the treatment of municipal and industrial water supplies. The primary reason for its recent adoption in wastewater treatment has been the need to upgrade effluents from conventional treatment plants. Installations may use direct filtration of activated sludge or trickling filter effluents, without the addition of chemical agents. Also, deep-bed filters are employed in systems for phosphorus removal from secondary effluents and in physical-chemical systems for the treatment of raw wastewater. In these latter cases chemical coagulation, flocculation and sedimentation precede the filters as in water treatment plants.

In essence, the unit process of deep-bed filtration encompasses exhaustion of the bed followed by a regeneration. Water containing suspended solids is passed through a bed of granular material resulting in deposition of the suspended solids in the bed. Eventually the pressure drop across the bed becomes excessive or the ability of the bed to remove suspended solids is impaired. Thereupon filtration is stopped and the bed is cleaned prior to being placed back in service.

9.2 Filter Design

At the present time virtually all deep bed filters utilized for waste treatment are "rapid sand" type, i.e., downflow, static bed, with batch or semicontinuous operation. In this section, design information on this traditional type of filter will be reviewed. In a later section of this chapter, descriptions and design information for some new concepts in deep bed filtration will be given.

Figure 9-1 (3) is a cut-away view of a typical "rapid sand type" filter, illustrating most of its components. In essence the filter is a box containing filter media, an underdrain system, a backwash system, flow control systems and various conduits for bringing feedwater and wash water to and conveying filtrate and used wash water away from the filter. Figure 9-2 illustrates a pressure filter. It can be seen that there is little difference between the pressure and gravity flow filters except for the pressure housing. Because of size restrictions on pressure filters they are equipped with a simpler wash water collection trough system than gravity filters.

9.2.1 Design Parameters

Process design of a filter includes determination of:

a. type and size of filter media

b. depth of filter

c. rate, duration and timing of water backwash, air scour and surface wash

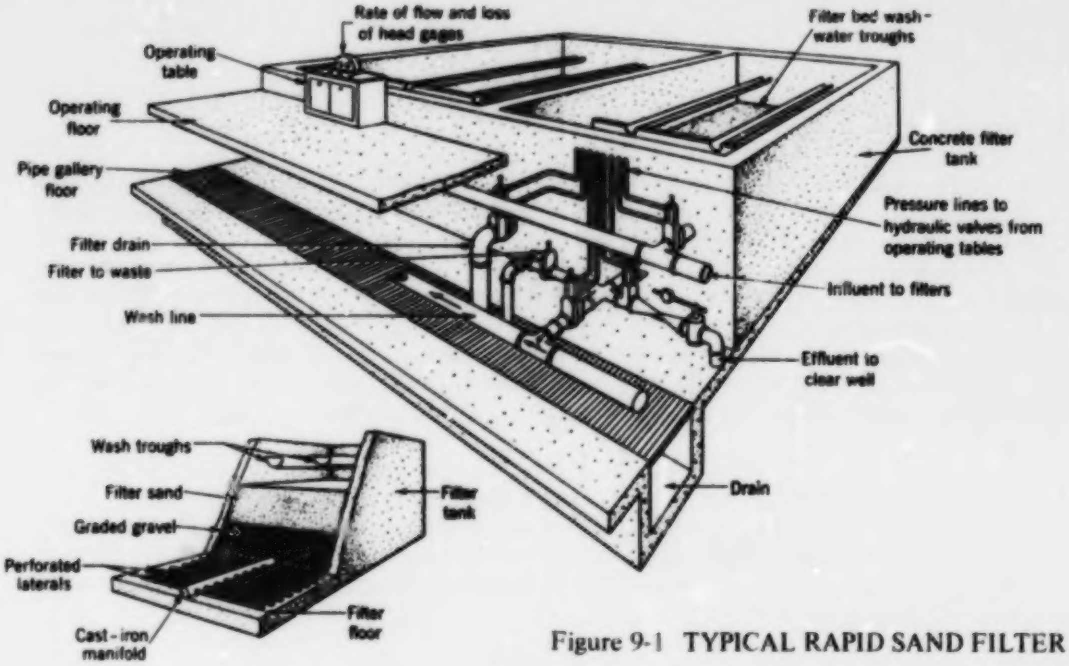
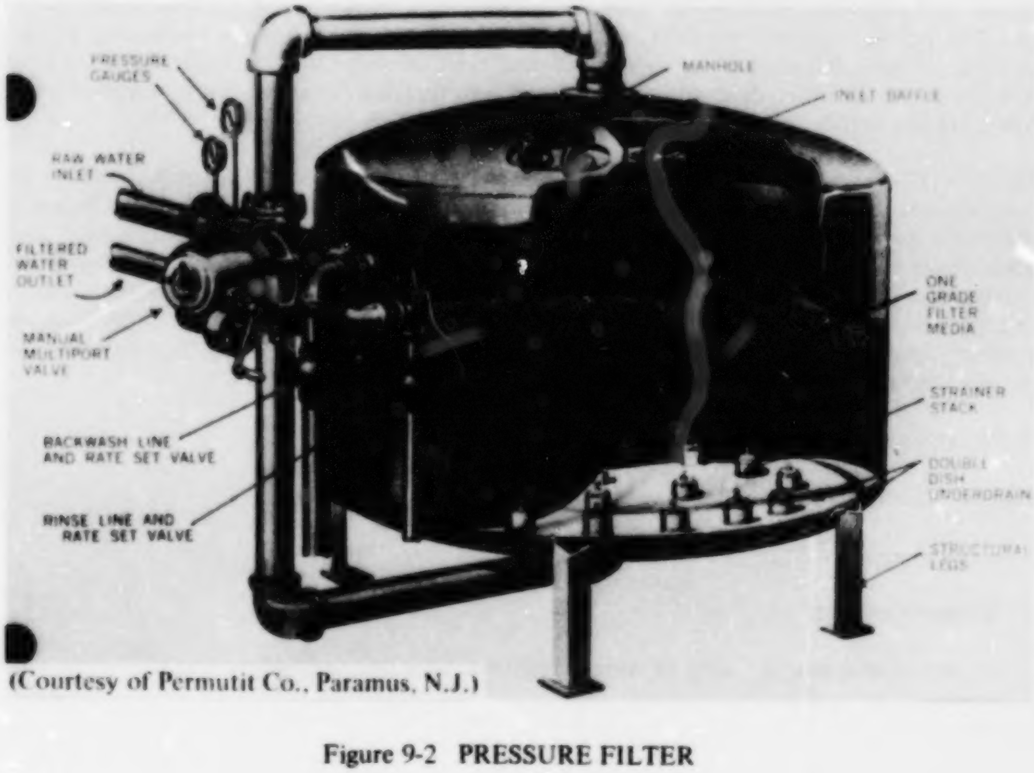

Rate of flow and loss of head gages
Filter bed wash-water troughs
Operating table
Operating floor
Concrete filter tank
Pipe gallery floor
Pressure lines to hydraulic valves from operating tables
Filter drain
Filter to waste
Influent to filters
Wash line
Effluent to clear well
Wash troughs
Filter sand
Filter tank
Drain
Graded gravel
Perforated laterals
Filter floor
Cast-iron manifold

Figure 9-1 TYPICAL RAPID SAND FILTER


PRESSURE GAUGES
MANHOLE
INLET BAFFLE
RAW WATER INLET
FILTERED WATER OUTLET
MANUAL MULTIPORT VALVE
ONE GRADE FILTER MEDIA
STRAINER STACK
BACKWASH LINE AND RATE SET VALVE
DOUBLE DISH UNDERDRAIN
RINSE LINE AND RATE SET VALVE
STRUCTURAL LEGS

(Courtesy of Permutit Co., Paramus, N.J.)

Figure 9-2 PRESSURE FILTER

d. filtration rate

e. type of chemical pretreatment and dose requirement

f. expected duration of filter runs.

Values for each of these parameters must be selected to yield a design which will produce an effluent of the desired quality at a minimum cost.

The difficulty in arriving at an optimum process design is twofold. Many, if not all, of these parameters are interdependent. Also, the present level of filtration theory can only semi-quantitatively represent the interdependence of the process design parameters.

Process design information should be generated from pilot plant data. At best, present filtration theory can only reduce the degree of pilot plant study required. All too often process design of a filtration installation is performed according to "rule of thumb" experience. Use of such a procedure usually results in an overdesign (uneconomical) or an underdesign (system failure to meet quality or quantity objectives.)

9.2.2 Pilot Plant Studies

A rather large number of interdependent parameters must be considered in pilot plant studies. In addition, the range of variation of each which may have to be investigated is quite large. Thus, a field pilot investigation cannot generally be of short duration. Pilot studies up to a year in length may be required, but it should be possible to decrease this time period by utilizing experience as a guide in establishing limits of variation of the parameters.

Pilot studies can be physically described as consisting of a series of filter columns using an available supply of feedwater. Provisions must be made to measure effluent quality with time, pressure drops over various sections of the bed and over the entire bed with time and backwash rates required to achieve specified degrees of media expansion. The studies should be conducted at several flow rates, with several media types and size ranges, to various terminal headloss values and with a variety of chemical pretreatments where appropriate.

The data resulting from these studies should be utilized to eliminate from further consideration those situations which obviously cannot meet effluent criteria or cannot be justified economically. Based on these analyses, a narrowed-down set of parameters can be used in a second set of investigations. This second study may be more comprehensive than the first series, including such variables as expected diurnal variation and slight changes in chemical dose. This process may continue through several more rounds before an "optimum design" is achieved. This final design should be given a lengthy test (one month or more) at conditions as close as possible to those expected in the field. Even after this lengthy study, it is probable that only a good approximation of field conditions will be obtained.

Specific details for conducting filtration pilot studies have not been established. The following is a list of equipment and practices usually used:

a. Multiple filter tubes of transparent material with a minimum diameter of 2 to 3 in. are utilized.

b. The tubes are fitted for either gravity or pressure operation.

c. A false bottom underdrain is utilized with either a porous plate or strainer backwash system.

d. Flow control is established by the use of a positive displacement pump or an effluent throttle valve connected to a float.

e. Pressure taps are provided above and below the media, as well as at other locations within the bed. Tap locations are generally located near the top of each type of media used.

Additional details of pilot filters are given in various references on filtration studies (1), (2).

Two areas of process design cannot be as adequately explored in pilot studies as the others listed above. These are the parameters associated with cleaning the bed and the effect of chemical treatment of the feed. Cleaning of the filter bed is difficult to simulate in pilot scale because of the small surface area of the beds utilized. The small area makes it impossible to study surface wash and air scour. Results of water backwash may not be representative because of the wall effect. Chemical treatment of the filter feed without flocculation and sedimentation may be simulated in the pilot studies. If flocculation and sedimentation are employed after chemical treatment however, the performance of model flocculators and clarifiers may not closely simulate that of full-scale field units. Thus the feed to the filters may not be representative. Both areas of difficulty can be overcome by using large pilot installations, i.e., minimum filter size of about 2' x 2' to study backwash, and minimum clarifier size of about 10' diameter to simulate full-scale sedimentation. However, going to this scale is not usually economically justified. If studies are being conducted at an existing plant, i.e., one that has full-scale clarifiers or filters, advantage should be taken of the situation.

Although pilot studies give the most reliable information on which to design filters, it should be remembered that filtration is a mixture of art and science. Excessive fine tuning of the process design should not be attempted. The chosen process design should provide a flexible system which will do the job with an adequate safety factor at a reasonable cost. Fine tuning of the process should be left to the plant operator.

9.2.3 Filter Media

Selection of the size, type and depth of the filtration media is the single most important decision in the design of a filtration system. Unfortunately, it is impossible to specify optimum media characteristics from theoretical considerations. Pilot studies should be conducted with several sizes and types of media prior to making a decision.

Sand has historically been the filtration medium most commonly used, but anthracite coal and, to a lesser extent, garnet have been employed. These substances occur in nature and are used in filters in a graded size range. Although use of a uniform size medium would have certain advantages, it is not economical to excessively restrict the size range.

A typical grain size distribution curve for a naturally occurring filter medium is given in Figure 9-3. This curve is often a straight line on log-probability paper. Historically, the two points used to characterize a medium are the 10 percent size and the 60 percent size. These

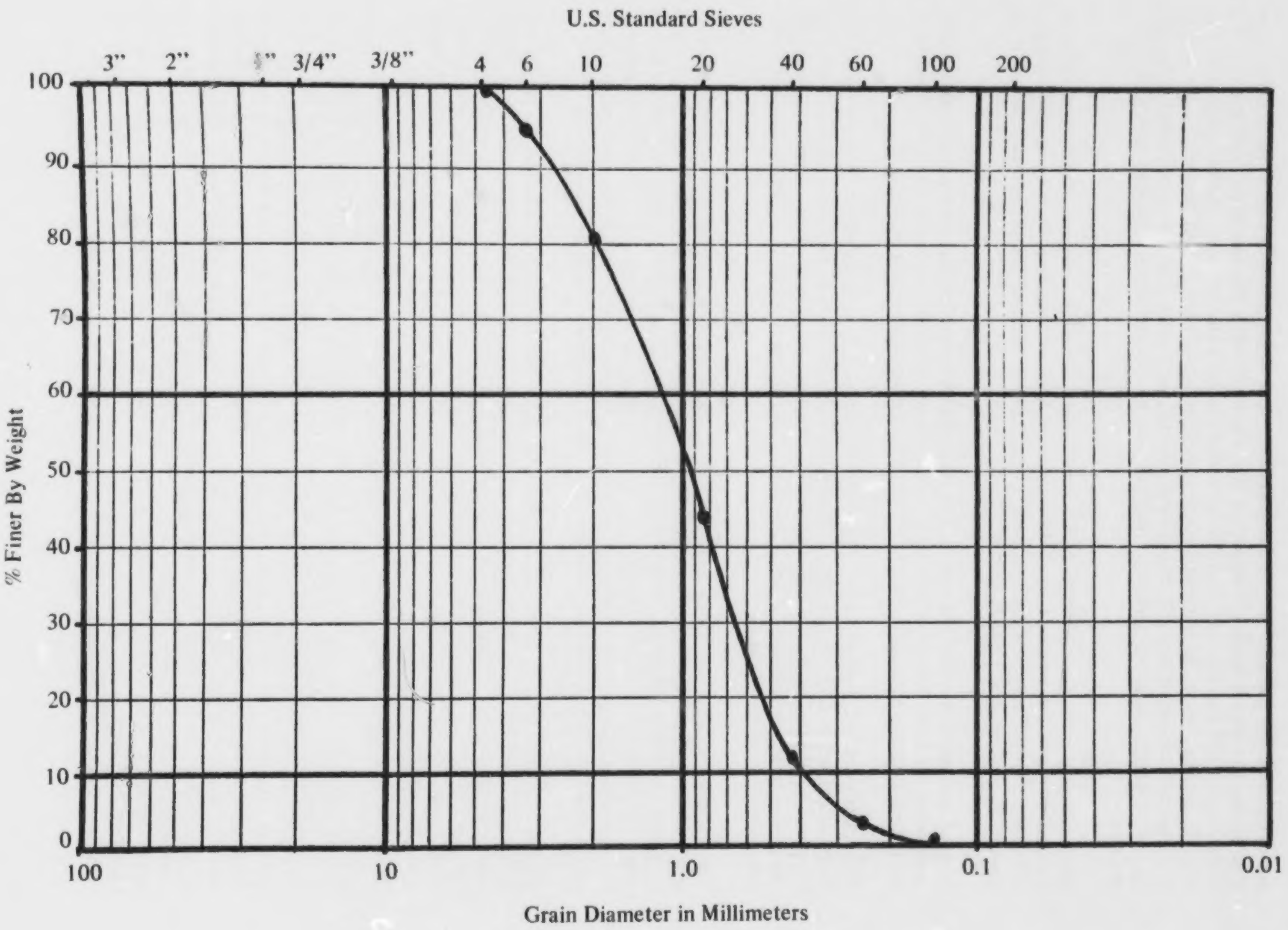


Figure 9-3 GRAIN SIZE CURVE

are defined as the particle size of a distribution such that the weight of all smaller particles constitutes the stated weight fraction of the whole. Usually the specification is given as the effective size (10 percent size) and the uniformity coefficient (ratio of the 60 percent size to the 10 percent size). In general, good correlation has been found between clean water headloss and effective size (3). Fair and Geyer (3) present a method of calculating the size fractions which must be sieved or washed out to convert from one size distribution to another.

Until recently most filter designs called for a single filtration medium, either sand or coal. Typical effective sizes range from 0.35 mm to 0.8 mm with uniformity coefficients between 1.3 and 1.7 (4). These size specification ranges can be utilized for preliminary selection of sizes to be evaluated in pilot studies.

In coal and sand media of identical size, it has been found that the solids removal with coal is somewhat inferior, but the rate of pressure drop build-up is lower (5). This finding is explained by the greater angularity of coal with consequent greater porosity.

The relationship of medium size to filter performance can be generalized. Smaller media are more effective in removing suspended solids at the expense of increased pressure drop or headloss buildup rates, i.e., shorter filter runs. Larger media have reduced initial headlosses and pressure drop increases during the filter run, but may yield a higher suspended solids concentration in the effluent.

The purpose of pilot studies is to quantify the above experiences for the wastewater under study. The quantitative relationships developed must then be used in conjunction with economic and physical design factors to define the optimum design. The historic trend has been toward coarser sizes of filter media in order to attain higher flow rates without reducing the length of filter runs. In order to assure an adequate effluent quality with coarse media and high filtration rates, chemical treatment of the feed may be required.

Another recent trend in filtration has been the adoption of the multi-media concept. Conventional single medium filters have a fine to coarse gradation in the direction of flow which results from hydraulic gradation during backwash. This type of gradation is not efficient as virtually all of the removal and storage must take place in the upper few inches of the filter with a consequent rapid increase in headloss. A coarse to fine filter gradation is much more efficient as it provides for much greater utilization of bed depth, using the fine media only to remove the finer fraction of the suspended solids.

One method of obtaining a coarse to fine filter gradation is the dual-media filtration concept. This employs the use of a layer of coarse anthracite coal over a layer of fine sand. The sizes of the anthracite and sand are chosen so that the coarser but lighter anthracite (specific gravity 1.6) will remain above the heavier (specific gravity 2.65) but smaller sand during backwash. It is desirable to have the coal as coarse as possible to prevent surface blinding and the sand as fine as possible to promote high degrees of removals. However, the disparity in sizes cannot be too great lest overtopping of the coal by the sand would result. In general, sand sizes much finer than 40 mesh are not utilized because the coal size required to prevent overtopping by sand during backwash would be too small to allow high filtration rates. To ascertain the degree of mixing which will occur during backwashing and its effects on subsequent filter performance, pilot column studies are best utilized.

An extension of the dual media concept is the mixed-media filter. Ordinarily this is a tri-media filter of coal over sand over garnet (specific gravity 4.2). There is evidence that judicious selection of the size of each medium will allow a degree of intermixing of the media, such that a reasonable approximation of continuous coarse to fine gradation is obtained. Figure 9-4 illustrates the conventional single medium filter, a non-mixed dual media filter and an ideal coarse-to-fine filter. Typical designs of multimedia filter beds are given in Table 9-1.

Table 9-1

TYPICAL MULTI-MEDIA DESIGNS (6)

Design No.	Garnet		Sand		Coal	
	Size (Mesh)	Depth (Inches)	Size (Mesh)	Depth (Inches)	Size (Mesh)	Depth (Inches)
1	-40x80	8	-20x40	12	-10x20	22
2	-20x40	3	-10x20	12	-10x16	15
3	-40x80	3	-20x40	9	-10x20	8

Conley and Hsiung (6) present additional information on the design of multi-media filters for a variety of applications.

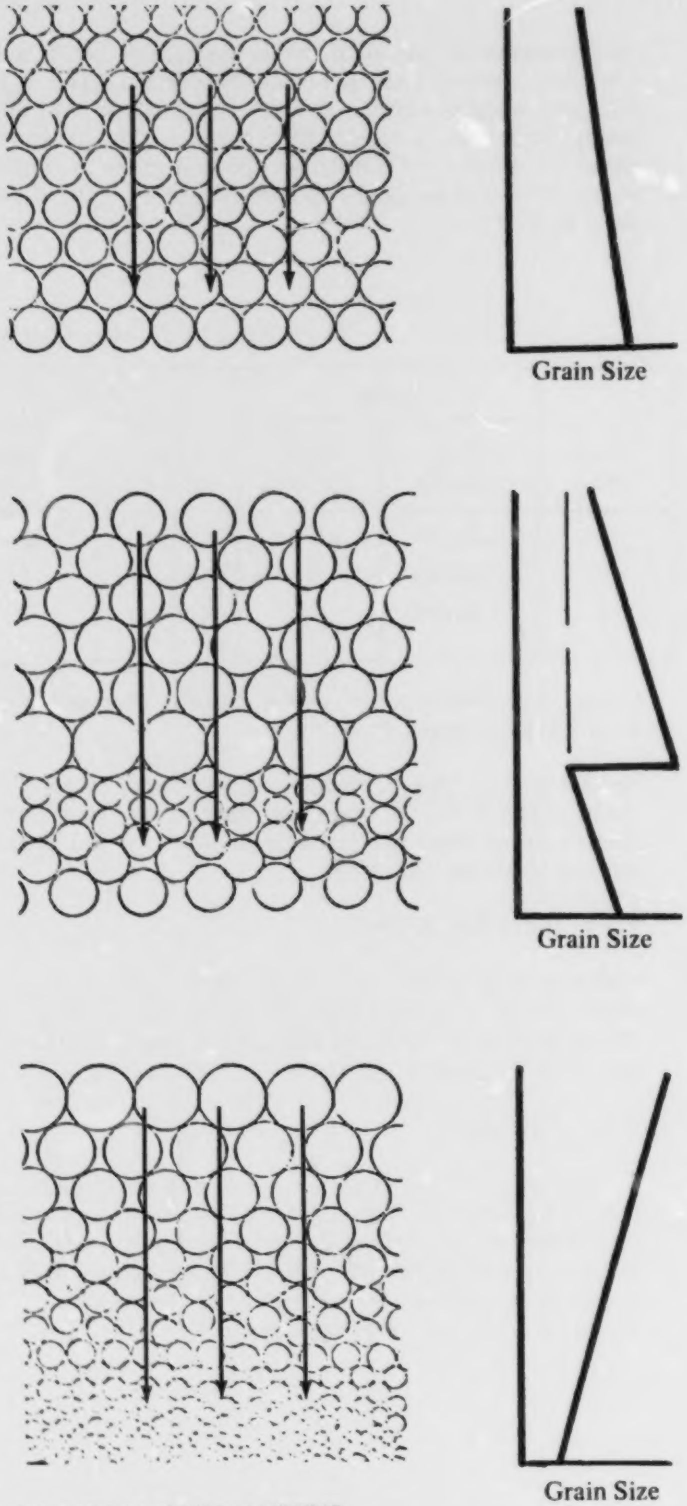
Several studies have compared the performance of single medium, dual media and multi-media filters (7), (8), (9). These studies indicated that in general the latter two types of filters outperformed single medium filters. Better effluents were obtained at higher flow rates, with longer filter runs.

9.2.4 Filter Bed Depth

Although it is known that single medium filters make effective use of only the top few inches of the bed, the historical practice of designing deep beds (24 to 36 in.) has not been abandoned. Dual and multi-media filter depths of the same magnitude are justified because the full filter depth is utilized. In fact, deeper filters of this type may find utility.

9.2.5 Flow Rates

Traditionally, the design flow rate of most single medium filters was 2 gal/min/ft^2. Recently these have been raised to 4 gal/min/ft^2 or greater when coarser media were employed along with higher terminal pressure drops. The multi-media filters have been successfully operated at rates up to 8 gal/min/ft^2 on a continuous basis. Rates of this magnitude or higher can be anticipated for design in the future. Of course, pilot studies are essential in determining the design rates of a full-scale installation.


Cross-Section Through
Single-Media Bed
Such as Conventional
Rapid Sand Filter

Cross-Section Through
Dual-Media Bed
Coarse Coal Above
Fine Sand

Cross-Section Through
Ideal Filter
Uniformly Graded From
Coarse to Fine
From Top to Bottom

Grain Size
Grain Size
Grain Size

Figure 9-4 MEDIA COMPARISONS

9.2.6 Filter Size, Layout and Housing

Both gravity and pressure filter systems of standard sizes are provided by various filter manufacturers. Specific size details can be obtained from manufacturers' catalogues. Gravity filters are usually constructed on site of concrete. Common feedwater and backwash water supply header pipes generally travel down a central pipe gallery (see Figure 9-1) between rows of filter elements. The gravity filters are usually enclosed in a filter building with storage of finished water in the basement area. All pumps, motors, controls, etc., are housed in the filter building.

Pressure units are more individualized than gravity filters. They are fabricated of steel and are cylindrical in shape. Structural design is according to standard pressure codes. Vessels up to 10 feet in diameter and 60 feet in length are available. It is essential that a manhole be incorporated in the design to allow for maintenance. The filter should be designed with a means for hydraulic removal of all the filter media. Sight glasses for observation of the bed should also be incorporated in the design.

9.2.7 Filter Cleaning Systems

Termination of a filter run takes place when either of these events occurs:

a. The effluent does not meet the quality criteria

b. The bed pressure drop is excessive

Either event indicates that the filter is excessively dirty and must be cleaned. The importance of an effective cleaning system on the filter system performance has been reviewed in several recent publications (10), (11). If effective cleaning is not obtained, short filter runs and poor effluent quality will result. Indeed in many respects, improper cleaning sets up self perpetuating operational difficulties such as mud balls, filter media slime coating and media cracking (5).

Backwashing the filter bed by flow reversal is the major, and in many cases, the only method used to clean the bed. The sources of backwash water may include feedwater, filter effluent or some other effluent downstream of the filter. It is virtually impossible to predict the specific conditions required to insure cleaning of the bed by this technique. Experience indicates that for a single medium filter provision should be made to backwash the bed for a 10- to 15-minute period at a rate which will insure fluidization of all the media. The optimum requirements can only be determined by the plant operator once the plant is in service. The plant operator should be instructed that excessive expansion of the bed is just as bad as insufficient expansion. Optimum expansion varies with the size of the medium, type of floc and penetration of floc.

The most important aspect of the design of the backwash system is to insure uniform distribution of the wash water over the entire cross section of the filter area. Although the function of the underdrain system of the filter is both to collect the filtrate and distribute wash water, the latter function controls the design.

A variety of underdrain systems are available for use. The traditional system employs several layers of graded gravel under the filter bed with a lateral header system positioned on the filter floor. The lateral header system is equipped with orifices and provides the preliminary distribution of the wash water. The final distribution is accomplished as the water moves upward through the gravel. Culp & Culp (12) provide rules for the design of a lateral header system.

Several commercial systems are available which employ patented false bottom distributors in place of the lateral header system. The gravel is then placed on top of the false bottom. Diagrams of Leopold Block and Wheeler Filter Bottom systems are illustrated in Figure 9-5.

Some systems have been devised with the concept of eliminating the gravel layers. One system utilizes porous plates over a false bottom, as shown in Figure 9-5. Another system also utilizes a false bottom with strainers on 12" centers. The strainers are nozzles fabricated of plastic and metal with many small openings.

Table 9-2 presents the usual specifications for gravel, as well as a new design suggested by Baylis (5).

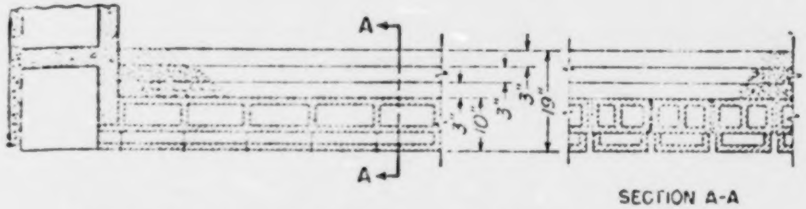
Table 9-2

FILTER GRAVEL DESIGN

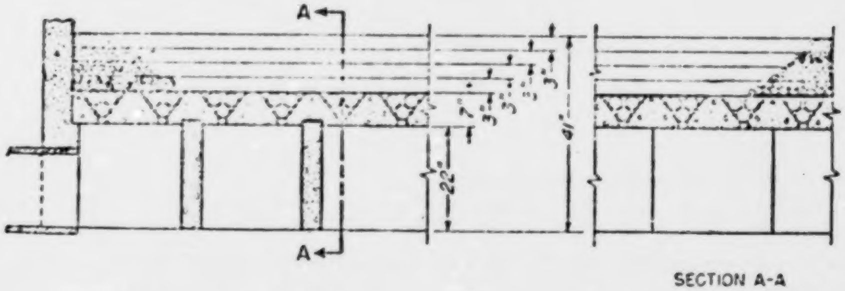
Standard		Baylis	
Depth	Size	Depth	Size
2-1/2"	1/12"-1/8"	5"	1" - 2"
3-1/2"	1/8"-1/4"	2"	1/2"-1"
3-1/2"	1/4"-1/2"	2"	1/4"-1/2"
2"	1/2"-3/4"	4"	1/8"-1/4"
4"	3/4"-1-1/2"	2"	1/4"-1/2"
6"	1-1/2"-3-1/2"	2"	1/2"-1"
		4"	1" - 2"

It was found that with the old design the upper layers of gravel could fluidize if excessive backwash rates were used. The Baylis design, which places a final heavy layer of gravel over the finer gravel, prevents this fluidization.

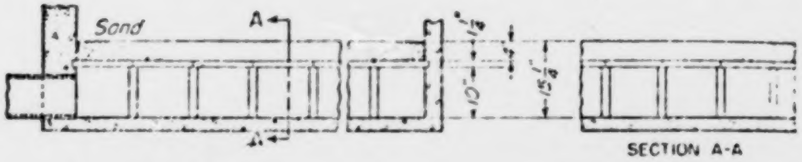
After moving upward through the expanded bed the wash water is conducted out of the filter by wash water troughs. In a pressure filter the inlet baffle serves as the wash water collector. In a gravity filter separate troughs are used because of the large area which must be serviced. In order to prevent non-uniform flow of wash water, the maximum lip to lip distance between troughs cannot exceed six feet. The troughs are usually rectangular channels. A convenient width is assumed and the depth computed by a momentum analysis of the backwater curve in the trough. Figure 9-6 is a nomogram for this solution. For example,


A
A
3"
10"
3"
3"
19"
SECTION A-A

A Leopold-block underdrain, with glazed-tile blocks furnishing passages to the water, instead of using laterial pipes.


A
A
3"
3"
3"
3"
7"
41"
22"
SECTION A-A

The Wheeler-filter bottom, consisting of solid, inverted, truncated pyramids with water connections at the apex of each pyramid, and the pockets filled with cement or glazed earthenware spheres.


Sand
A
A
1½"
10"
15¼"
SECTION A-A

Porous-plate filter bottoms.

Figure 9-5 FILTER BOTTOM AND UNDERDRAIN SYSTEMS

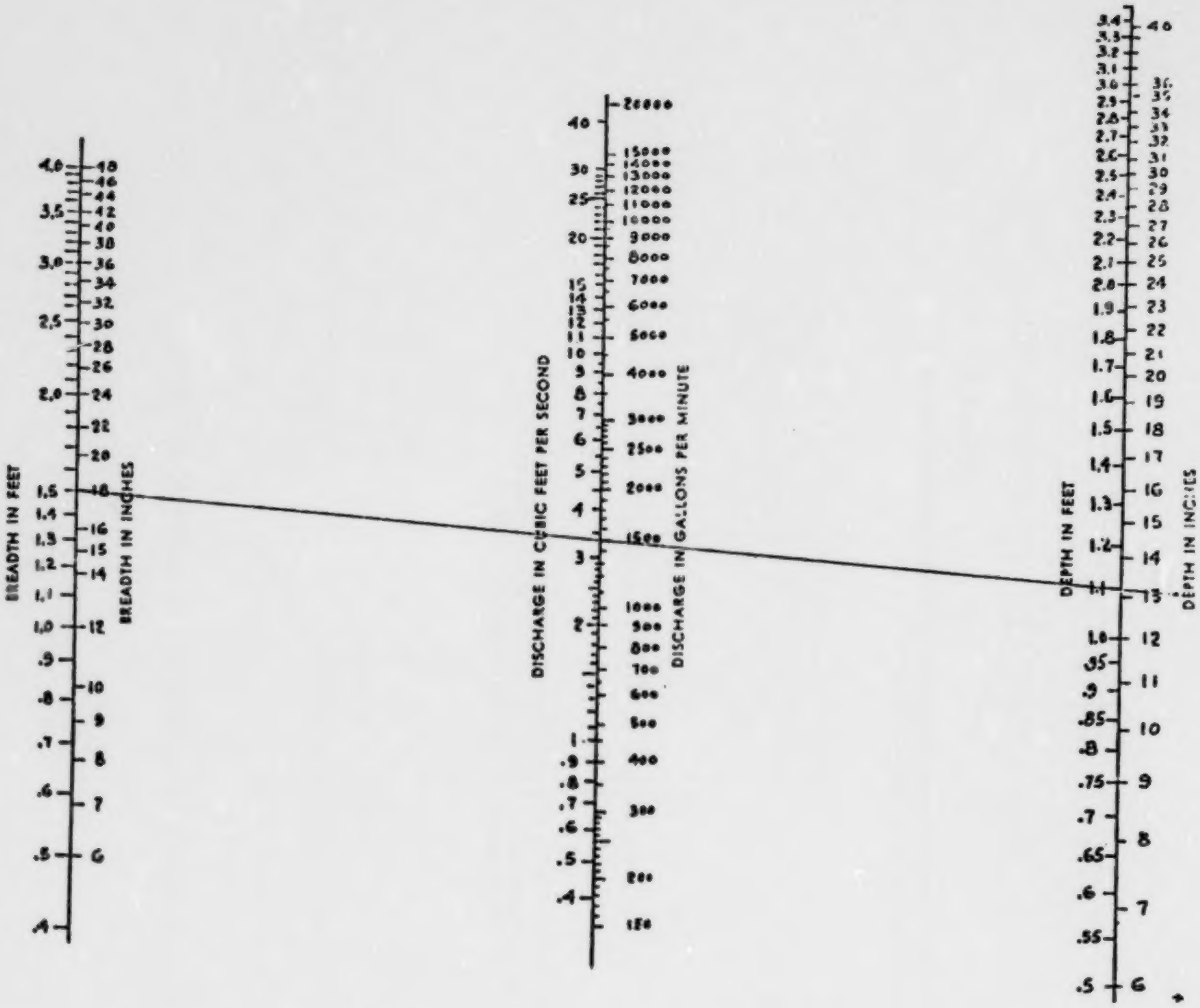


Figure 9-6

NOMOGRAM FOR SELECTING WASH TROUGH DIMENSIONS an 18 inch wide trough carrying a flow of 3.33 ft^3/sec will have a maximum water depth of 13.2 inches. The side wall depth should be increased by 2 to 3 inches to allow for freeboard.

Maintaining a clean bed by backwash alone has been found to be difficult when wastewater, even after biological and/or chemical treatment, is the filter feed. Even periodic chlorination during backwashing has not always been successful in preventing slimes. Consequently, either surface wash or air scour should be employed in waste treatment filters. Multi- and dual-media filters allow for greater floc penetration of the bed than single medium filters. The use of surface wash and/or air scour is mandatory under these circumstances. In essence, the function of air scour and surface wash is to loosen the accumulated deposits in the filter. The normal backwash then flushes the deposits away.

A surface wash apparatus is illustrated in Figure 9-7. Either fixed nozzles or rotating pipes fitted with nozzles are placed about 1 to 2 inches above the top of the bed. While the surface wash is on, the backwash expansion is set at a lower rate than after the surface wash is terminated. Surface wash water is supplied at 50 to 100 psi at rates approximating 1 to 3 gal/min/ft^2 of bed.

Air scour is accomplished by injecting air into the underdrain system prior to initiating water backwash. The following procedure has been recommended (12):

a. Stop influent and lower the water level to a few inches above bed.

b. Apply air alone at 2-5 ft^3/min/ft^2 for 3-10 min.

c. Apply water backwash at 2-5 gpm/ft^2 with air on until water is within one foot of wash water trough.

d. Shut air off.

e. Continue water backwash at normal rate for usual period of time.

f. Apply backwash for 1-2 min. at a rate required to insure hydraulic classification of the filter media.

Air backwash has the disadvantage of increased possibility for coal media losses. Although small losses repeatedly occur due to air bubble attachment during normal backwashing cycles, the danger of massive losses exists if air is applied during trough overflow periods. Control is especially difficult in pressure filters as it is hard to observe the bed during backwash.

9.2.8 Filter Control Systems

Recent improvements and advances have been made in the systems which are used to control the operation of filters. As a result, where once a filter required a manual operation, it is now possible to have a completely automated filtration plant. Not only have these advances removed much of the drudgery from filter operation, but they have given the plant operator powerful techniques for improving filter performance.

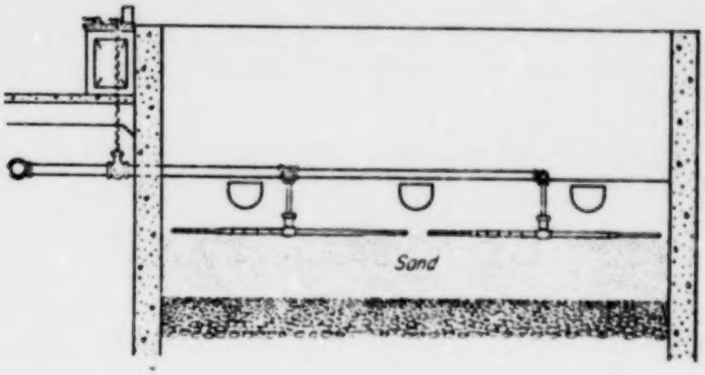


Figure 9-7
PALMER FILTER BED AGITATOR

Perhaps the most important of the recent advances is the use of automatic turbidimeters to continuously monitor the filter feed and product. This allows the operator to anticipate difficulties from changes in feed quality, and rapidly remedy process failures. In addition, these devices allow the operator to rapidly evaluate the effects of changes in process variables and provide a continuous record of plant performance. All turbidimeters operate on the principle of measurement of scattered or transmitted light. A variety of commercial instruments are available.

Automatic backwash, initiated either by terminal pressure drop across the bed or by excess turbidity in the effluent, should be considered for all new plants. The cost of these systems may be justified by a reduction in labor. Automatic backwash systems are available from a variety of manufacturers. It is important that these systems be equipped with delay mechanisms so that the wash water reservoir capacity is not exceeded if the wash cycles of several filters overlap.

The automatic backwash system referred to above is essentially an electronics package which operates valves, pumps, etc., by remote control. Two other automatic backwash systems based on a different concept are commercially available. An automatic gravity filter is depicted in Figure 9-8. No data are available for waste treatment application of these systems.

A traveling backwash filter employs an 11-inch depth of sand supported on porous plates. The bed is divided into many sections 8 inches wide. A traveling backwash assembly moves from section to section as required to clean each section. Thus most of the filter is in operation with only a small section being washed at any time.

In the past, one type of flow control system was utilized in most filters. This system provides for constant flow with a constant water depth over the filter. Under this condition, constant flow is maintained by varying the headloss downstream of the filter so that the total headloss in the filter and downstream is constant. Headloss in the downstream line is automatically adjusted by a throttle valve connected to a preset counterweight. Usually a venturi with a variable opening diaphragm serves as the rate controller. Frequently maintenance of these rate controllers is troublesome.

An alternate system which does not require a rate controller to achieve constant flow has recently been described (13). A weir in the inlet channel to the clear well provides a constant back pressure on the filter. Flow into the filter box is over an inlet weir with the crest set quite high above the bed surface. As headloss develops in the filter, the depth of flow above the filter increases to maintain the filter flow constant. A disadvantage of this method is the capital cost associated with building the filter box walls several feet higher than required where a rate controller is used.

Another control concept is declining-rate filtration. This method is applicable only to medium or large scale plants which utilize multiple filters. With this method the flow rate through the filter is allowed to decline as the filter clogs. The filters are staggered in degree of clogging so that the total production of the plant is constant. This procedure is claimed to produce better effluent quality and longer filter runs because flow slows as the filter clogs. Thus, the rate of headloss increase decreases and the probability of floc breakthrough is lessened.

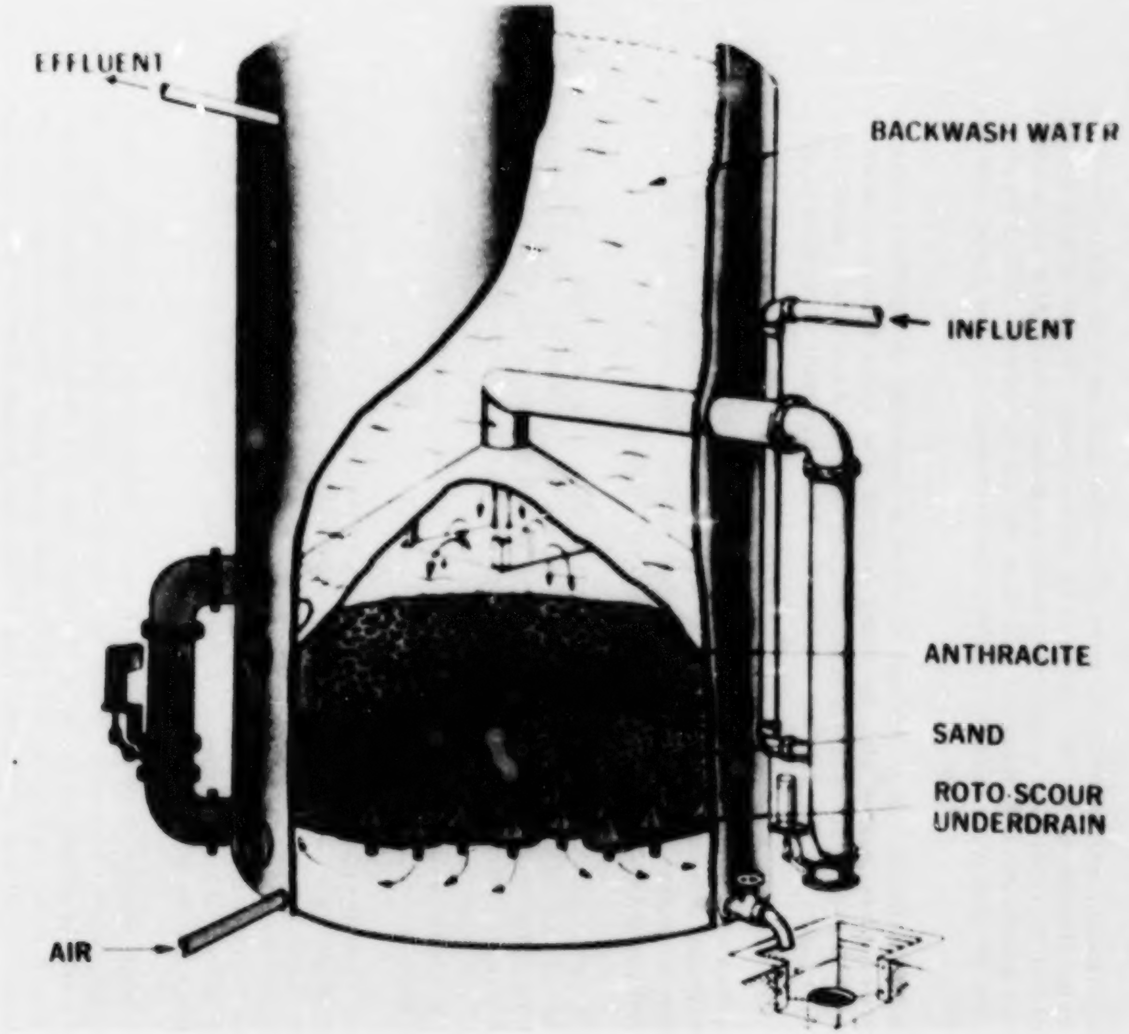

EFFLUENT
BACKWASH WATER
INFLUENT
ANTHRACITE
SAND
ROTO-SCOUR UNDERDRAIN
AIR

Figure 9-8 AUTOMATIC GRAVITY FILTER, SINGLE COMPARTMENT
(Courtesy of Graver)

A simple method of flow control which can be used with pressure filters is to pump at a constant rate to each filter with a positive displacement pump. With this system, the pump rate sets the flow rate and the pump discharge pressure rises as the bed clogs.

9.2.9 Chemical Pretreatment

Chemical pretreatment ahead of the filter is designed to:

a. Coagulate suspended solids to make them more amenable to sedimentation and/or filtration.

b. React with soluble components which must be removed to form insoluble precipitates for removal by sedimentation and/or filtration.

c. Adjust the strength of the floc to control the degree of penetration of solids into the filter.

In order to ascertain optimum chemical doses to accomplish any of these objectives, pilot studies are required.

While coagulants aid in removal of colloids, the floc formed may be relatively weak. The major use of polymeric materials is to strengthen floc so that it will not penetrate through the filter. The floc should not be excessively strengthened or it will not penetrate beyond the filter surface, producing excessive pressure drop. A good rule to follow is that floc breakthrough should coincide with the achievement of terminal headloss.

Chemicals are usually applied prior to sedimentation to remove the bulk of the solids before the filter. Recent practice has employed further chemical addition just prior to filtration to enhance the filterability of the feedwater solids.

9.3 Summary of Results of Filtration Studies

Rapid sand type filters have been used in three kinds of waste treatment systems: direct filtration of secondary effluent, filtration of secondary effluent after chemical treatment and filtration of raw or primary wastewater after coagulation and sedimentation. Representative results from field studies in each of these areas will be discussed below.

9.3.1 Filtration of Secondary Effluent

Early work on filtration of secondary effluent took place in Europe. Truesdale and Birkbeck (14) reported on tests run between October, 1949, and May, 1950, at the Luton Sewage Works. Beds of sand 2 feet deep, ranging in size from 0.9 mm to 1.7 mm, exhibited 72 to 91 percent removal of suspended solids and 52 to 70 percent removal of BOD. Flow rates ranged from 1.33 to 3.3 Imp. gal/min/ft^2. Air-scour-aided backwash was used once per day to clean the bed. Backwash flow rate was 11.6 Imp. gal/min/ft^2.

Naylor, Evans and Dunscome (15) later reviewed 15 years of studies of tertiary treatment at Luton. A 3-foot deep bed of -10 to +18 mesh sand consistently provided an effluent of 4 to 6 mg/l suspended solids at flow rates of 3.3 Imp. gal/min/ft^2. It was found best to wash the beds every 12 hours.

In the U.S., most direct filtration work has been with activated sludge feed. At the Hyperion Plant in Los Angeles, sand of 0.95 mm effective size was used in a shallow bed (11 inches deep) traveling backwash filter. This study lasted for six months during which time 46 percent suspended solids removal and 57 percent BOD removal were obtained. Filtration rate was 2 gal/min/ft^2. Difficulty was encountered in cleaning the filters and performance gradually deteriorated during the study. Use of a finer sand (0.45 mm effective size) in an attempt to yield a better effluent was a failure due to very rapid clogging of the filter (16).

Much greater success utilizing the traveling backwash filter for activated sludge effluent treatment was obtained by Lynam (17) in Chicago. The effective size of sand used in this study was 0.58 mm. Suspended solids removal of 70 percent and BOD removal of 80 percent were obtained at flow rates of 2 to 6 gal/min/ft^2. Terminal headloss was quite low (11 inches of water.) The range of flows studied exhibited no significant difference in terms of suspended solids removal.

A study of filtration of activated sludge effluent was presented by Tchobanoglous and Eliassen (18). They found that flow rate (2 to 10 gal/min/ft^2) had little effect on performance, but effective size of sand in the range of 0.4 mm to 1.2 mm had a very significant effect. It was also determined that virtually all suspended solids removal took place in the upper six inches of the bed. It was concluded that the floc strength of activated sludge is quite high on the basis of the low degree of penetration even with very coarse media and high flow rates. Poor removal of suspended solids (10 to 40 percent) was obtained in this study, with filter depths greater than 6 inches producing no additional benefits. It was found that a bimodal distribution of particle sizes existed for the activated sludge effluent. Apparently, only the larger solids could be removed by the filters.

Culp and Culp (12) reviewed the work on plain filtration of secondary effluent with both single medium and multimedia filters. They concluded that, with either type of filter, better results would be obtained as the degree of self flocculation of the sludge increased. Thus, a high-rate activated sludge effluent which contains much colloidal material should filter poorly, while an extended aeration effluent should filter well. Multi-media filters exhibit a marked superiority for filtration of activated sludge effluent because of the high volume of floc storage available in the upper bed and the polishing effect of the small media. They indicated the expected performance of multi-media filters for plain filtration of secondary effluents, as shown in Table 9-3.

Table 9-3

EXPECTED EFFLUENT SUSPENDED SOLIDS FROM MULTI-MEDIA FILTRATION OF SECONDARY EFFLUENT

Effluent Type	Effluent S.S. mg/l
High Rate Trickling Filter	10 - 20
2-Stage Trickling Filter	6 - 15
Contact Stabilization	6 - 15
Conventional Activated Sludge	3 - 10
Extended Aeration	1 - 5

9.3.2 Filtration of Chemically-Treated Secondary Effluent

Treatment of secondary effluent by chemical coagulation, sedimentation and filtration has been conducted at a number of installations. The purposes of this treatment procedure have often been twofold, suspended solids and phosphorus removal. Unfortunately, in many cases, performance results have been reported for complete systems, not each individual process. Thus, analysis of the filter performance is not possible. In most of these installations, the filter is viewed essentially as a polishing device to capture solids which escape the sedimentation tank. As Culp and Culp (12) have indicated, this philosophy may be wrong, as modern filters can absorb much greater solids loads than older designs.

In conjunction with the studies at Chicago (17), coagulation with alum followed by filtration was evaluated. It was found that the alum treatment had little effect on the effluent quality. It is probable that this result is due to the weakness of chemical floc compared to activated sludge floc.

An advanced waste treatment plant has been used to renovate step aeration activated sludge effluent in Nassau County, New York. Alum at 200 mg/1 is used to coagulate the wastewater prior to sedimentation and filtration. The filters are dual-media containing 30 inches of 0.9 mm coal over 6 inches of 0.35 mm sand. With the addition of 0.5 mg/1 of an anionic polymer, effluent turbidity is maintained below 0.4 JTU. Run length varies from 8 to 24 hours depending on the solids load from the clarifier.

At Lebanon, Ohio, treatment of the activated sludge effluent with lime has been investigated (19). The lime dose averaged 300 mg/1. Dual-media filters, consisting of 18 inches of 0.75 mm coal over 6 inches of 0.45 mm sand, followed clarification. The filters were operated at 2 gal/min/ft^2, and were backwashed when the leadloss reached 9 ft of water. Influent to the filters ranged from 13 to 36 mg/1 of suspended solids (turbidity 4 to 10 JTU.) Filter effluent ranged from 0.07 to 0.14 JTU.

At Lake Tahoe,lime is used to coagulate activated sludge effluent prior to sedimentation and filtration (12). Multi-media beds 3 feet deep are utilized at an average flow rate of 5 gal/min/ft^2. Filter runs have varied from 4 to 60 hours. Polymers as well as alum have been used to strengthen the floc. Normally, runs are terminated at headlosses of 8 ft of water. Effluent turbidities are typically reduced to 0.3 JTU with correspondingly low values for other parameters.

At the Environmental Protection Agency pilot plant at Washington, D.C., mineral addition to the final phase of a step aeration activated sludge is practiced for phosphorus removal. After sedimentation the effluent is filtered through parallel filters at 2.4 gal/min/ft^2. One filter is a dual-media, while the other is a multi-media. It has been found that the multi-media filter removes 5 to 10 percent more suspended solids than the dual-media filter. Filter runs have been in the range of 24 to 32 hours. Typically, the filters reduce secondary effluent suspended solids from 33 mg/1 to 8 mg/1 (20).

9.3.3 Filtration Following Chemical Treatment of Primary or Raw Wastewater

Clarification of raw wastewater followed by carbon adsorption is just emerging as a viable treatment technology. This system employs filtration as part of the solids separation system. Although several pilot installations employing this concept are in operation, filtration has not been closely studied; thus data are sparse.

At the EPA pilot plant in Washington, D.C., two-stage lime treatment of raw sewage followed by sedimentation, filtration and granular carbon adsorption is being studied (21). Dual-media filters (18 inches of 0.9 mm coal over 6 inches of 0.45 mm sand) are employed. Cleaning is initiated at a headloss of 9 feet of water. Cleaning is performed automatically with a surface wash rate of 3 gal/min/ft^2 and an upflow rate of 20 gal/min/ft^2. Run lengths averaged 50 hours during cold weather, but in warm weather the growth of slimes reduced run lengths to less than 12 hours. Prechlorination of the filter feed was employed to restore the run lengths to 50 hours. Filter effluent averaged 4.5 mg/l of suspended solids over a 6-month period, which represented a 70 percent efficiency for the filter. This plant operates on a programmed diurnal flow variation which produces a flow rate variation on the filter from 1.7 to 4.3 gal/min/ft^2.

A similar system using single-stage lime treatment was run for several months at the EPA installation in Lebanon, Ohio (22). Flow to the filters was 2 gal/min/ft^2. Suspended solids in the filter effluent averaged 10 mg/l, which represented a 67 percent removal efficiency.

9.4 New Filtration Systems

The rapid sand type filtration system previously discussed has been a downflow, batch, static bed system. Consequently, it suffers from a variety of process deficiencies including:

a. The need to stop the process periodically to clean the filter medium.

b. The limited ability to economically handle suspensions containing high concentrations of suspended solids.

During the last decade a number of new filtration systems have been developed which are aimed at overcoming these shortcomings. Several of these will be discussed below.

9.4.1 Upflow Filtration

As indicated previously, a major difficulty with downflow, single-media filtration is that after backwash, the bed is graded fine to coarse in the direction of flow. If filtration is conducted upflow, this difficulty is circumvented. However, once the headloss produced by the upflow exceeds the buoyant weight of the filtration medium, fluidization with consequent loss of filtration efficiency will result. One solution to this problem is to place a restraining grid on or near the top of the filter medium to prevent fluidization. A diagram of an upflow filter is illustrated in Figure 9-9 (courtesy of DeLaval.)

The spacing between bars of the grid must be large enough to allow the bed to expand during backwash, but must be small enough to prevent upward bed movement during filtration. It would seem that these two requirements are directly contradictory; however, arching of the grains takes place between the bars, allowing a reasonably large spacing. Space between the bars is usually in the range of 100 to 150 diameters of the smallest grain size in the beds. During cleaning, air is first introduced to agitate the bed. After the air has broken the arches, backwashing with water is started. Table 9-4 gives a summary of typical design parameters for the upflow filters.

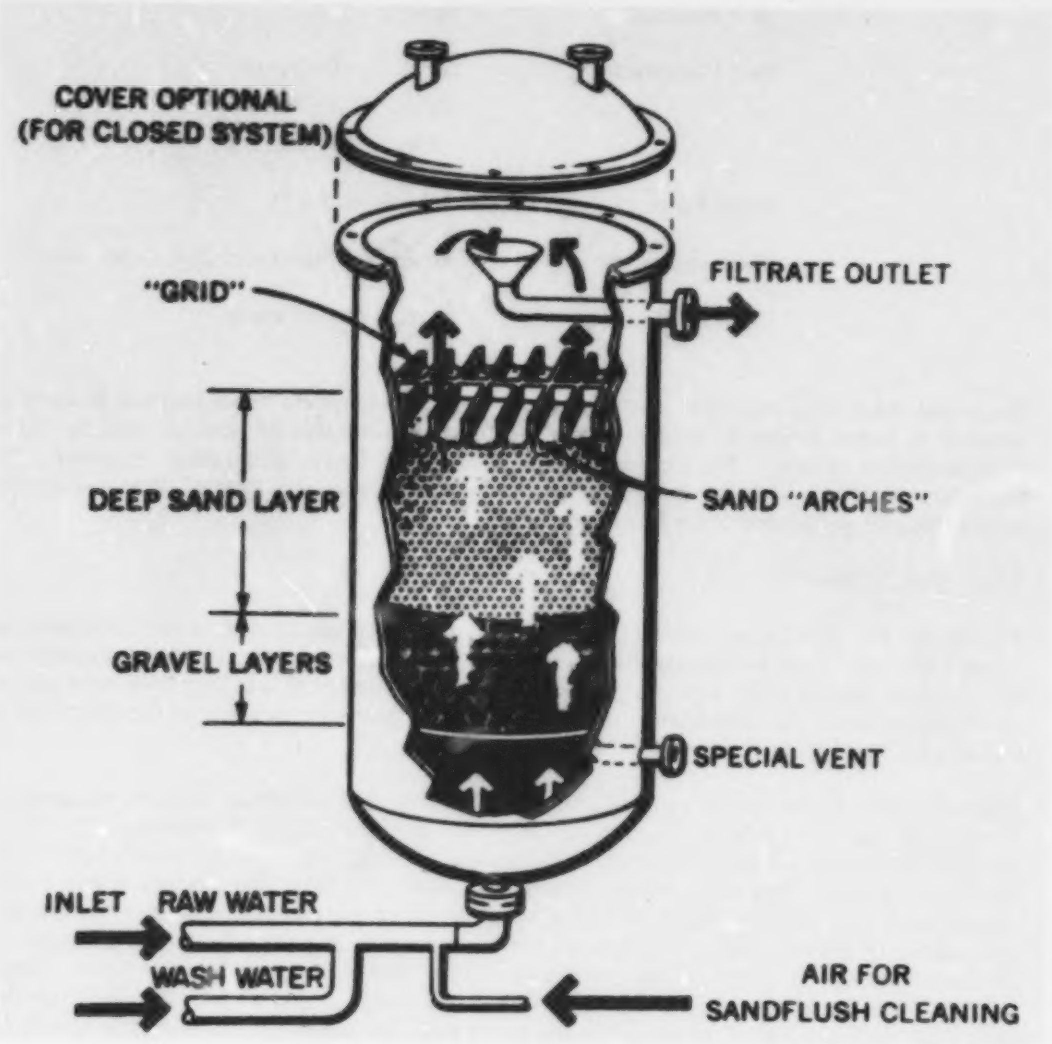

COVER OPTIONAL
(FOR CLOSED SYSTEM)
FILTRATE OUTLET
"GRID"
DEEP SAND LAYER
SAND "ARCHES"
GRAVEL LAYERS
SPECIAL VENT
INLET
RAW WATER
WASH WATER
AIR FOR
SANDFLUSH CLEANING

Figure 9-9 CROSS SECTION OF UPFLOW FILTER

Table 9-4

DESIGN PARAMETERS FOR UPFLOW FILTERS

Bed Material:	Sand	
Bed Construction:	60 in.	1 - 2 mm
	10 in.	2 - 3 mm
	4 in.	10 - 15 mm
Flow Rate:	2-3 gal/min/sq ft	
Backwash Rate:	To achieve minimum 20% expansion	
Terminal Headloss:	6 to 20 feet of water	

Boby and Alpe (23) reported on the performance of an upflow filter treating secondary effluent at Totor, England. Average suspended solids removal was 85 percent, with the filter effluent below 5 mg/1. These results were equal to or better than those obtained with downflow filtration with the same size bed. It is claimed that this type of filter can absorb higher loads of solids than a conventional filter.

9.4.2 Moving Bed Filter

The basic concept of a moving bed filter is the mechanical movement of the most heavily clogged portion of the medium out of the zone of filtration with virtually no interruption of the filtration process. The potential of such a process is operation at higher flow rates and at much higher solids loadings than conventional systems. Superior cleaning of the filter media should also be possible.

Johns-Manville Corporation has developed a moving bed filtration system. A diagram illustrating the essentials of this system is given in Figure 9-10. Wastewater (A) flows through the inlet pipe where chemicals, if required, are added at (B). The wastewater enters the head tank (C) and then passes through the sand bed (D). The filtered water leaves through the exit screens. When excessive headloss develops, the bed is pushed toward the head tank by pressurizing a chamber separated from the bed by a flexible diaphragm. A mechanical cutter (F) sweeps down over the face of the bed cutting off the top layers. These then fall into the hopper (G) of the head tank. The sludge and sand are removed from the head tank with the aid of an ejector using feedwater. The solids are hydraulically conveyed to the sand washer (H) where filtered water or air and filtered water are used to backwash the sand. Clean sand moves by gravity back to the base of the filter. The spent washwater is sent to a sedimentation tank for removal of the wastewater solids. The operation of the system is automated.

Under an Environmental Protection Agency contract (24), this system was evaluated by the manufacturer for the treatment of raw wastewater, primary effluent and settled and unsettled trickling filter effluents at the Bernards Township Sewage Treatment Plant. Alum was used to precipitate phosphorus, and an anionic polymer was employed to prevent excessive floc penetration. The results, given in Table 9-5, show excellent treatment performance in these situations.

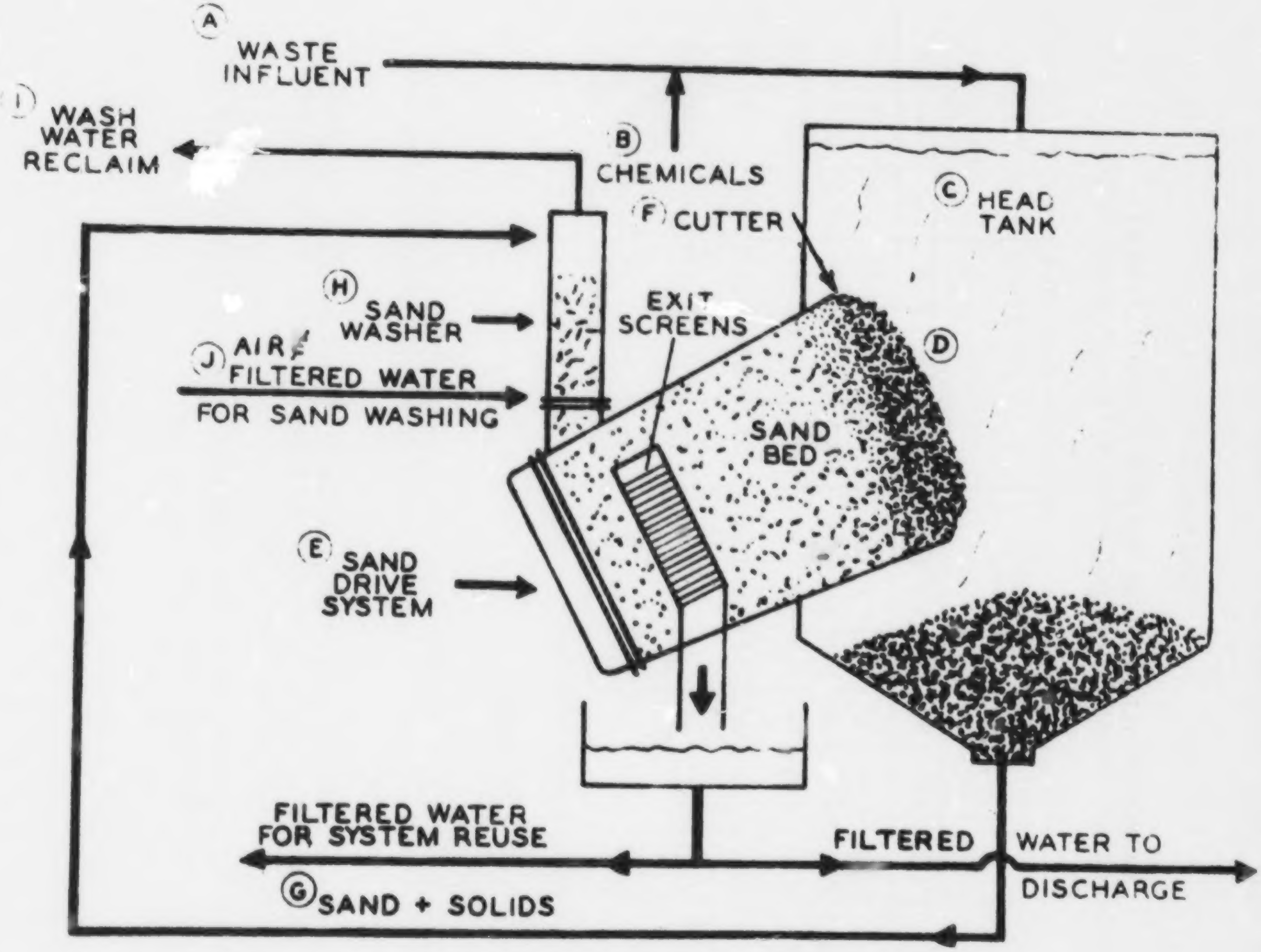

Ⓐ WASTE INFLUENT
Ⓘ WASH WATER RECLAIM
Ⓑ CHEMICALS
Ⓒ HEAD TANK
Ⓕ CUTTER
Ⓗ SAND WASHER
EXIT SCREENS
Ⓓ
Ⓙ AIR / FILTERED WATER FOR SAND WASHING
SAND BED
Ⓔ SAND DRIVE SYSTEM
FILTERED WATER FOR SYSTEM REUSE
FILTERED WATER TO DISCHARGE
Ⓖ SAND + SOLIDS

Figure 9-10

SCHEMATIC DRAWING OF THE JOHNS-MANVILLE MOVING BED FILTER

The moving bed filter is being evaluated at full scale (2 MGD) at the Borough of Manville Sewage Treatment Plant, Manville, N.J. At this site, unsettled trickling filter effluent is the feed. This study is being partially funded by an EPA demonstration grant.

Table 9-5

JOHNS-MANVILLE MOVING BED FILTER EVALUATION AT BERNARDS TOWNSHIP SEWERAGE AUTHORITY TREATMENT PLANT

Parameter	Final Effluent w/o Chlorination			Unsettled Trickling Filter Effluent			Primary Effluent			Raw Wastewater		
(mg/l)	In	Out	%	In	Out	%	In	Out	%	In	Out	%
Total P	9.37	0.51	95	19.1	0.99	95	14.6	1.13	93	21.5	2.16	91
Filterable P	8.03	0.11	99	14.9	0.62	96	13.2	0.58	96	18.6	0.79	96
Ortho P	7.80	0.10	99	12.4	0.53	96	9.8	0.38	96	13.2	0.57	95
BOD$_5$	65	12	80	55	3.8	93	67	12	82	115	19	84
Suspended Solids	50	15	70	86	7.1	91	77	11	87	156	27	83
Turbidity (JTU)	33	7	79	39	3.4	91	53	3.7	93	123	16.7	87

Alum: 200 mg/l (commercial grade)

Polyelectrolyte: 0.5 mg/l anionic

The moving bed filter system is currently available in modules of 2-bed and 4-bed configurations. These modules can be used singly or in combination to accommodate large flow requirements as necessary.

a. The flow rate through any unit or combination of units is dependent on the quality of the incoming liquid and the discharge requirements. Flow rates up to 7.0 gal/min/ft^2 of exposed filter area are possible.

b. Filter bed dimensions - 48-inch face diameter, 60-inch length from the face of the unit to the center line of exit screen.

c. Module dimensions -

	2-Bed Unit	4-Bed Unit
Plan length	20'	20'
Plan width	7'	14'
Height	17'	17'
Weight, empty	22,000 lb	44,000 lb
operating	78,000 lb	156,000 lb
Plan floor space, module	140 ft^2	280 ft^2
system	350 ft^2	720 ft^2

d. Power requirements -

Function	Two-Bed System Conn. HP	Two-Bed System Op. HP	Four-Bed System Conn. HP	Four-Bed System Op. HP
Diaphragm or Sand-bed Movement	3.0	3.0	6.0	6.0
Cutter	3.0	0.75	3.0	1.5
Sand Cleaning System	3.75	3.75	7.5	7.5
Screen Wash	7.5	0.5	7.5	1.0
Chemical Pump and Mixer	1.0	0.5	1.0	0.75
Air Compressor	1.0	0.5	1.0	0.75
	19.25	9.0	26.0	17.5

e. Filter media - Hard sharp-grained quartz sand of filter grade quality (effective size 0.6 to 0.8 mm; uniformity coefficient 1.5)

f. Sand drive rate -
Linear - 12 in./hour - maximum
Volume - 25 ft^3 /hour - maximum
Pump pressure - 75 psi - average
150 psi - maximum

g. Head tank - 30 minute detention time

h. Sludge settling tank - 30 minute detention time

i. Controls -
Low level and high level off-on
Differential pressure to actuate bed movement and cutter
Effluent turbidity to adjust chemical feed

9.4.3 Radial Flow Moving Bed Filter

Recently, Dravo Corporation has introduced a radial flow moving bed filter to the American market. At present, most applications of this system have been for industrial waste treatment. In addition to the moving bed concept, the main feature of this system is the radial flow concept. This geometric configuration provides more filter area per unit volume than downflow or upflow systems. As the liquid flows radially from the central core, it slows down, providing increased opportunity for solids removal.

9.4.4 Radial Flow-External Wash-Filter

The Hydromation Corporation has developed a new concept in filtration. A diagram of this filter is illustrated in Figure 9-11. It is a batch-type radial flow filter with external media wash. When a filter run is terminated, the media is pumped out of the filter and upward into a scrubber. The flow velocity in the scrubber is 20 ft/sec, which produces a very high degree of turbulence, assuring good cleaning of the media. The clean media circulates around the backwash loop to the radial flow bed.

This filtration system utilizes a polymer resin as the filter medium. It is claimed to have superior dirt holding capacity compared to natural media. Because of the superior dirt holding character and special cleaning system, flow rates of 10 to 20 gal/min/ft^2 have been purported.

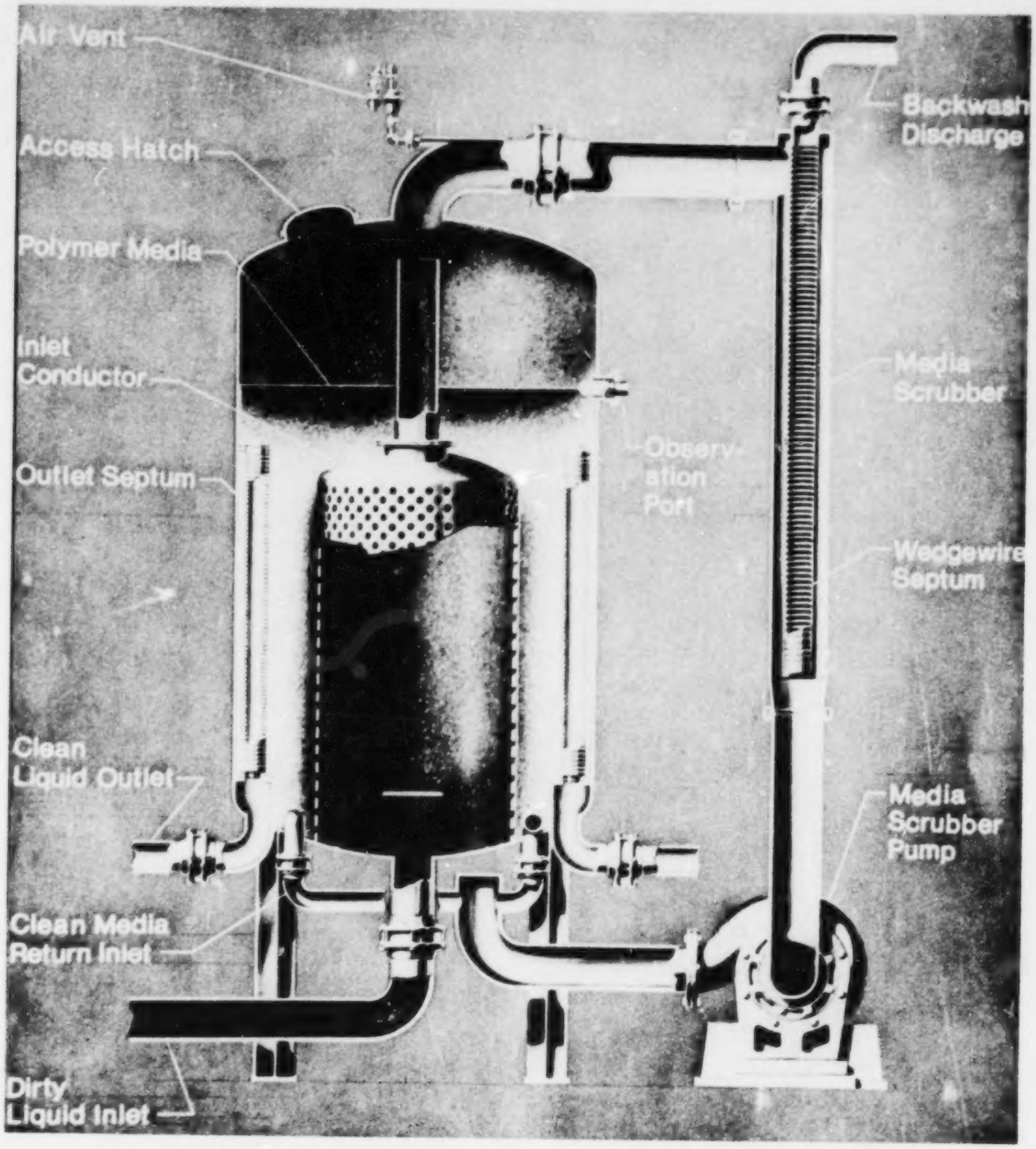

Air Vent
Access Hatch
Polymer Media
Inlet Conductor
Outlet Septum
Clean Liquid Outlet
Clean Media Return Inlet
Dirty Liquid Inlet
Backwash Discharge
Media Scrubber
Observ-ation Port
Wedgewire Septum
Media Scrubber Pump

Figure 9-11 HYDROMATION IN-DEPTH FILTER

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CHAPTER 10

OPERATION AND MAINTENANCE

10.1 General

This section presents some of the operation and maintenance factors that must be considered in the use of solids removal processes. Particular interest is paid to man-hour requirements for processes by consideration of the work required. First, general practices and procedures that are common to several or all of these processes are presented. Operation and maintenance (O/M) requirements peculiar to the individual processes will follow. Finally, an estimate of man-hour requirements for proper operation and maintenance of these processes will conclude this section. Only the more feasible processes on which sufficient information is available are specifically discussed.

10.2 General O/M Considerations

Any process, insofar as it requires housing for protection of equipment and employees, will also require regular custodial services. These include cleaning and washing of floors and walls, the washing of windows and the disposal of refuse. These services are often overlooked or disregarded in the operation of a waste treatment facility, since their performance does not contribute directly to process operation and maintenance. However, proper attention to these services is essential to create a suitable working environment and a positive mental attitude on the part of the employees. Areas that serve primarily as process enclosures may be cleaned on a weekly basis. Administrative areas, laboratories and workshops should be cleaned daily.

Those processes which employ exterior basins must be provided with means for rapid dewatering to expose submerged equipment and structures for inspection, servicing and repair. While the frequency for performance of these activities varies considerably depending on the number of process units and their size, an average frequency of at least twice a year is recommended. At these times, the basin equipment should be thoroughly cleaned. All exposed cracks in structural concrete should be filled and the exposed metal surfaces of structure and equipment should be cleaned and painted or touched up to prevent corrosion. Daily cleaning of weirs, channels, inlet boxes and outlet troughs is necessary to prevent paper, dirt, stones and other assorted debris from clogging these facilities and interfering with process efficiency.

A daily procedure for performing process monitoring and control functions should be established. This should include frequent visits to each unit process to observe its operation. Controls and gages should be checked weekly and calibrated on a regularly scheduled basis. Also, a review should be made of the most recent operational records and reports in order to stay abreast of the current operating problems. In addition, a daily sampling program for the effluent of each process is required to maintain proper process control. This will involve the collection of a series of composite and grab samples, either manually or by the use of automatic sampling devices. These samples must also be analyzed and the results tabulated on a regular basis to make the data available for reports and operational decisions.

The establishment of a daily program of equipment inspection and repair is an absolute operational requirement for the proper maintenance and operation of these processes. The complexity of the formal preventive maintenance (P/M) program will depend largely upon the size of the plant and the number and overall capabilities of assigned O/M personnel. The most essential areas to be covered are the inspection and repair of electrical and operating control systems; the greasing and lubrication of all mechanical wearing surfaces; the cleaning of pumps, valves, conduits, etc.; and the regular painting of all exposed metal or wood surfaces.

Electrical and control equipment should be inspected regularly for tightness and to see that moving parts are free, contact pressures firm, and shunts unfrayed. Controls should also be checked to see that they are operating at rated voltage. It is most important to make certain that these devices are clean and dry. Motors should also be cleaned and oiled on a regular basis and starter contact points inspected for proper fit and corrosion. All indicated repairs should be made at once.

Lubrication is one of the most important parts of a maintenance program. In this regard, strict adherence to the manufacturers recommendations should be practiced. It is important not to overlubricate, since this can also result in motor failure.

A certain amount of time should be spent each day in the preparation of records, forms and reports that describe the performance of the individual unit process and the overall treatment system. It is recommended that records be kept on:

- treatment efficiency
- influent and effluent quality
- volume of wastewater treated
- results of chemical analysis
- other additional data as required by control agencies
- equipment repair history
- inventory control data
- status of P/M program
- cost data:

    chemicals expended
    purchase of supplies
    man-hours expended
    power consumption

10.3 O/M Considerations - Individual Processes

The following is a list of operation and maintenance factors that are specific to the individual processes under consideration. Some of these processes are conventional and O/M data from their use in the treatment of water or wastewater are published. Some of the newer processes have not received large scale application at this time. The O/M factors for these processes can only be surmised based upon an analysis of the mechanics involved. This generally involves an anticipation of potential problems rather than a recording of past history.

10.3.1 Chemical Mixing and Flocculation

The major operational problems involved in this process are the receiving, storing, transporting and feeding of the chemicals used. Chemical feed pumps and bottom sludge pumps must be periodically disassembled and cleaned. As with any process that involves the agitation of domestic wastewater, a foam can occasionally develop in the rapid mix tanks and flocculators. This may be readily controlled by the application of surface sprays. Weir plates, baffles and drains must be kept clean to maintain proper basin hydraulics. Weir crests must always be kept sharp and clean. Motors, drive systems and paddles will require servicing and repair.

10.3.2 Solids-Contact Clarification

The operation and maintenance functions for this type of unit are very similar to those for the conventional process. The gain in spacial efficiency by the reduced size is balanced by increased maintenance requirements and costs due to the more intensive use of mechanical equipment. More frequent dewatering of the basins for inspection of submerged equipment will be required. More constant attention to the sludge removal function and the repair of these facilities is required to prevent excessive amounts of sludge from building up and interfering with efficient operations.

10.3.3 Sedimentation

The operation of a standard sedimentation tank does not usually require a great deal of time. Generally, the only piece of mechanical equipment employed will be the bottom scraper and the surface skimming mechanisms. Care must be taken to remove surface skimmings regularly to avoid carryover of the floating solids to subsequent units. The sludge collectors must be run often enough by manual or automatic timing devices to insure that solids do not build up to a point where the mechanism will be damaged or the effluent quality impaired. The side walls of the tank should be washed or scraped down on a regular basis so that solids do not collect on the surfaces and decompose causing an odor problem. Regular cleaning of weirs, troughs, skimmers and scrapers are required for effective performance. The scrapers of circular tanks should be leveled regularly to maintain proper balance. Weir crests should be kept sharp to maintain proper hydraulics.

Where more than one tank exists, care must be taken to be sure that the flow is properly distributed to each of the units. Adjustment at the influent structure must be made on a regular basis to be certain that each unit is receiving its share of the load.

Frequent samples of the sludge draw-off should be collected for laboratory analysis. A common problem is excessive draw-off of sludge which results in process water being pumped to the digesters with resultant upsets to that process. This can be controlled by regulation of the sludge pumps.

10.3.4 High-Rate Settlers

The operation and maintenance requirements of high-rate settlers should be similar to those of the conventional sedimentation basin. The size of the overall facility should be much smaller, but additional maintenance results from the regular cleaning requirements of the tubes themselves. Profuse biological growths occasionally occur in and on the top edges of the tubes, which may often be controlled by water sprays. Frequent basin drawdowns may be required if this method fails. Total man-hours required for operation and maintenance of this system should be similar to conventional sedimentation.

10.3.5 Microscreening

The operation of a microscreen installation requires regular checking of screen condition and headloss through the screen. Periodic adjustment of the ultraviolet lamp system as well as backwash pressure and flow rate are necessary. Periodic adjustment to the variable speed drum drive is needed to meet changing flow conditions if no automatic controls are used. The discharge weir must be raised or lowered to control screen submergence.

The major maintenance task is the cleaning and repair of the screens. If the unit must be stopped, it should be carefully drained and the fabric cleaned prior to removal from service. The drum should never be left standing for any length of time in a tank of dirty water. This can lead to severe clogging of the microfabric. This condition may be difficult to treat subsequently. Application of microscreening to chlorinated wastewaters can damage the screens by chlorine attack. Screen replacement should be performed on a regular basis and as soon as any defect is noted. Minor repairs can be made without removing the screen from the unit, and some screens have removable panels that can be replaced without affecting adjacent panels. Major screen defects call for the removal of the entire screen for repair or replacement on other models.

Other maintenance factors are the service required by the water spray heads and the ultraviolet light system. The ultraviolet system is used to control microbial slimes that often clog the screens. When the screen becomes heavily clogged, it is washed with a solution of sodium hypochlorite followed by a clean water rinse. This washdown will be needed periodically and requires about 1½ hours to complete. If the waste contains manganese or iron in solution, a film of iron or manganese oxide can be formed on the screen. This long term effect can be controlled by application of an inhibited acid cleaner in a manner similar to that with the hypochlorite.

10.3.6 Filtration

The major operating requirement for rapid sand filtration is the supervision of the backwashing operation. This operation must be closely controlled to be sure that the filtering media is kept free from dirt, hard spots, mud balls or other objectionable material. Often, normal filter backwashing is not enough to maintain the filter bed under all conditions. Regular inspection and maintenance are needed to insure that the beds remain loose, clean and free of incrustations. Frequent measurements of the depth of bed are taken and compared with previous measurements to be sure that the bed is neither expanding nor contracting excessively. This procedure may also help to determine media losses during intervening backwashes. Periodic testing of the level, slope and elevation of the wash water troughs is required to assure proper hydraulics. Regular painting of metal troughs is also important.

If the backwashing system is insufficient for complete cleansing of the bed, wastewater solids and chemical flocs may combine with media to form clumps or balls during the washing process. As their density increases, they sink to the bottom of the filtering media. Improper filtering and backwashing can result. In severe cases, it may be necessary to remove and replace the media.

10.3.7 Moving Bed Filter

The operation of a Moving Bed Filter requires continual surveillance because of the complexity of the unit. The complete system includes pumps for adding coagulating chemicals, advancing the sand filtering face, cleaning the discharge screen and operating the sand cleaning and feeding systems. The major maintenance tasks are servicing and repairing these various pumps.

It is important to service, on a regular basis, the chemical tanks, pumps and chemical feed lines. The sand cutting, washing and transporting systems and equipment have to be checked, serviced and repaired continuously due to the fact that they are handling a very abrasive material. This system must be maintained in prime condition or the process may break down and become inoperable. The diaphragm and pump assembly which actuates and advances the sand bed represents a critical phase of the operation as the pump is of a high pressure type with the attendant possibility of pipe and/or diaphragm failure. The pump used to develop the pressurized stream for cleaning the outlet screen from the sand filter is also of the high pressure type. Although used intermittently, this pump requires regular servicing, along with inspection of its associated connecting lines, fittings and nozzles. The air compressor and pump used in the agitation and washing of the sand require regular servicing. The flow and level controls, which compensate for variations in waste flow and composition, must be inspected and serviced regularly.

10.4 Man-Hour Requirements

In order to provide an estimate of the personnel required to perform the aforementioned operation and maintenance tasks, it is assumed:

a. That each unit process is part of a larger treatment system and operation and maintenance personnel can be shared with other units.

b. That most maintenance functions will be carried out by the operating agencies staff, and a negligible amount will be performed by outside contractors. Small plants may not be able to supply the complete range of specialties required. In these cases a regional maintenance system is postulated. This system would provide maintenance support to several plants.

c. That these processes are sophisticated enough to require 24-hour operation, seven days per week.

d. That maximum practical use be made of currently available automatic control systems. Therefore, the amount of time that the operating personnel spend in manual process control will be minimized.

e. That the basis of the detailed man-hour estimate will be to insure the efficient operation of a plant with a flow of 1 MGD.

Within the limits of these assumptions, total annual man-hours will be set by consideration of the aforementioned maintenance tasks, and the amount of time it should take reasonably skilled personnel to do the work. This rational approach would be greatly assisted by repair histories for these types of equipment from existing plants. However, because there are relatively few existing installations for most of these processes, this kind of information is not available. Because of the dearth of this information, these factors must be estimated based on knowledge of the application of these processes in other areas. These estimates are supplemented by manufacturers information. The following tables show the estimates of man-hours required for the individual unit processes considered. Table 10-1 is an estimated summary of man-hour requirements, showing a comparison between these estimates and others, where available. The remaining tables (10-2, -3, -4, -5, -6) show, for each individual process, the following:

- work categories
- job assignments
- performance frequency
- daily man-hours
- yearly man-hours

10.5 Job Descriptions

To aid in personnel recruitment, it is recommended that a set of job descriptions be prepared for each new facility. These descriptions should be tailored to the specific location, size and type of treatment process to be used. A list of standard job descriptions for wastewater treatment plant positions will soon be published by the Environmental Protection Agency. The following is a list of occupations commonly found in wastewater treatment plants:

Superintendent
Assistant Superintendent
Clerk Typist
Operations Supervisor
Shift Foreman
Operator II
Operator I
Automotive Equipment Operator
Maintenance Supervisor
Mechanical Maintenance Foreman
Mechanic II
Mechanic I
Electrician II
Electrician I
Maintenance Helper
Laborer
Painter
Storekeeper
Custodian
Chemist
Laboratory Technician

Table 10-1

MAN-HOUR SUMMARY[a]

By Process

Unit Process	Annual Man-Hours Net	Annual Man-Hours Gross[b]	Baseline[c] Man-Hours
Coagulation	2540	3180	2060
Sedimentation	1240	1550	–
Microscreening	2420	3030	540
Filtration	3875	4810	2260
Moving Bed Filter	3615	4520	N.A.

[a] Based on 1 MGD capacity

[b] Efficiency factor of 0.8 assumed, based on time lost for lunch breaks and in starting and finishing work.

[c] From curves in Reference (4), assuming 70% of direct O/M costs are for labor.

Table 10-2

MANPOWER REQUIREMENTS

MIXING AND FLOCCULATION

Task	Job Assignment	Performance Frequency	Daily Man-Hours	Yearly Man-Hours
Operation				
Process Monitoring	Operations	Daily	.75	275
Process Controlling	Operations	Daily	.50	180
Sampling	Operations	Daily	.50	180
Analysis	Chemist	Daily	1.00	365
Chemical Handling	Labor	As Required	–	400
0 – Subtotal				1400
Maintenance				
Custodial	Custodial	Daily		Neg.
Inspection & Testing	Maintenance	Daily	.25	90
Minor Parts Adjustment and/or Replacement	Maintenance	Daily	.50	180
Grease & Lubrication	Maintenance	Daily	.25	90
Major Overhaul and Cleaning	Maintenance	6 Mos.	–	150
Clean. (Weirs, Drains, etc.)	Labor	Daily	.50	180
Painting	Labor	As Required	–	100
Electrical	Electrician	As Required	–	300
M – Subtotal				1140
Total				2540

Table 10-3

MANPOWER REQUIREMENTS

SEDIMENTATION

Task	Job Assignment	Performance Frequency	Daily Man-Hours	Yearly Man-Hours
Operations				
Process Monitoring	Operations	Daily	.25	90
Process Controlling	Operations	Daily	.25	90
Sampling	Operations	Daily	.50	180
Analysis	Chemist	Daily	1.00	365
0 – Subtotal				360
Maintenance				
Custodial	Custodial	Daily	Neg.	Neg.
Inspection & Testing	Maintenance	Daily	.25	90
Parts Adjust. and Replacement	Maintenance	Daily	.25	90
Grease & Lubricate	Maintenance	Daily	.25	90
Major Overhaul	Maintenance	Daily		200
Cleaning (Weirs, Drains)	Labor	Daily	.50	180
Major Cleaning	Labor			80
Painting	Labor			50
Electrical	Electrician			100
M – Subtotal				880
Total				1240

Table 10-4

MANPOWER REQUIREMENTS

MICROSCREENING

Task	Job Assignment	Performance Frequency	Daily Man-Hours	Yearly Man-Hours
Operations				
Monitor	Operations	Daily	.75	270
Control	Operations	Daily	.75	270
Sampling	Operations	Daily	.75	270
Analysis	Chemist	Daily	1.25	450
[illegible] tal				1260
[illegible]				
[illegible]	Custodial	Daily	Neg.	Neg.
[illegible]on & Testing	Maintenance	Daily	.25	90
[illegible] djust.	Maintenance	Daily	.25	90
Mec[illegible] Repair & Replacement	Maintenance	As Required	–	200
Major Overhaul	Maintenance	6 Mos.	–	400
Media Cleaning	Maintenance	Bi-monthly	–	50
Control & Electric	Electrician	As Required	–	150
M – Subtotal				1160
Total				2420

Table 10-5

MANPOWER REQUIREMENTS

FILTRATION

Task	Job Assignment	Performance Frequency	Daily Man-Hours	Yearly Man-Hours
Operations				
Monitor	Operations	Daily	1.50	550
Control	Operations	Daily	1.00	365
Backwash Super.	Operations	As Required	2.00	730
Sampling	Operations	Daily	.75	270
Analysis	Chemist	Daily	1.25	450
0 – Subtotal				2365
Maintenance				
Custodial	Custodial	Daily	.25	90
Inspection & Testing	Maintenance	Daily	50	180
Parts Adjust.	Maintenance	Daily	.50	180
Major Overhaul	Maintenance	6 Mos.	–	200
Bed Maint. & Clean.	Labor	Daily	.75	280
Major Cleaning	Labor	6 Mos.	–	80
Controls & Elect.	Electrician	As Required	–	500
M – Subtotal				1510
Total				3875

Table 10-6

MANPOWER REQUIREMENTS

MOVING BED FILTER

Task	Job Assignment	Performance Frequency	Daily Man-Hours	Yearly Man-Hours
Operations				
Monitor	Operations	Daily	1.50	550
Control	Operations	Daily	1.00	365
Sampling	Operations	Daily	.75	270
Analysis	Chemist	Daily	1.25	450
Chemical Handling	Labor	As Required	–	250
Makeup Sand	Labor	As Required	–	100
0 – Subtotal				1985
Maintenance				
Custodial	Custodial	Daily	.50	180
Inspection & Testing	Maintenance	Daily	.75	270
Parts Adjust.	Maintenance	Daily	.50	180
Major Overhaul	Maintenance	Monthly	–	400
Control & Elect.	Electrician	As Required	–	600
M – Subtotal				1630
Total				3615

10.6 References

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CHAPTER 11

COST ESTIMATES

11.1 General

In general terms, suspended solids removal processes have an economy of size. In other words, larger capacity units evince lower treatment costs per unit volume of wastewater treated. This is due mainly to two factors, decreased capital costs per unit volume and decreased operating costs owing to more efficient usage of land, personnel and time. Although the actual values of these two factors varies for each process discussed, there is evidence that the above generalization is valid. Thorough treatises of costs for the suspended solids removal processes are available elsewhere (1), (2), (3). Most of the materials included in this text are from those sources. Care must be taken to denote the difference in amortization rates and periods and to adjust costs to current levels by use of available cost indices. Even after these normalizing adjustments are made, most estimates are not directly comparable due to differing geographical, administrative and design problems and varying assumptions. The most prudent use of the data provided is for guideline information on the relative costs of the processes and the variation in costs which might occur.

11.2 Process Costs

Figure 11-1 illustrates the estimated cost of a chemical clarification system utilizing solids-contact reactors (1). A recent estimate for two-stage lime clarification, excluding chemicals, shows costs varying from 11 cents/1000 gallons at 1 MGD down to 2.7 cents/100 gallons at 100 MGD (2). Evans and Wilson (3) indicates Tahoe costs for lime treatment, recalcination, recarbonation and sludge disposal to be 10.8 cents/1000 gallons for their 7.5 MGD facility. An October, 1970, estimate for two-stage lime clarification, recalcination, recarbonation and sludge disposal varies from 25.3 to 5.9 cents/1000 gallons for 1 and 100 MGD, respectively (4).

Capital costs for conventional sedimentation tanks (1) are shown in Figures 11-2 and 11-3. Barnard and Eckenfelder (5) related the cost of primary sedimentation to surface area using an overflow rate of 800 gpd/sf:

$$C = SA \left\{ 17.3 + \frac{6.7}{(SA)^{0.9}} \right\}$$

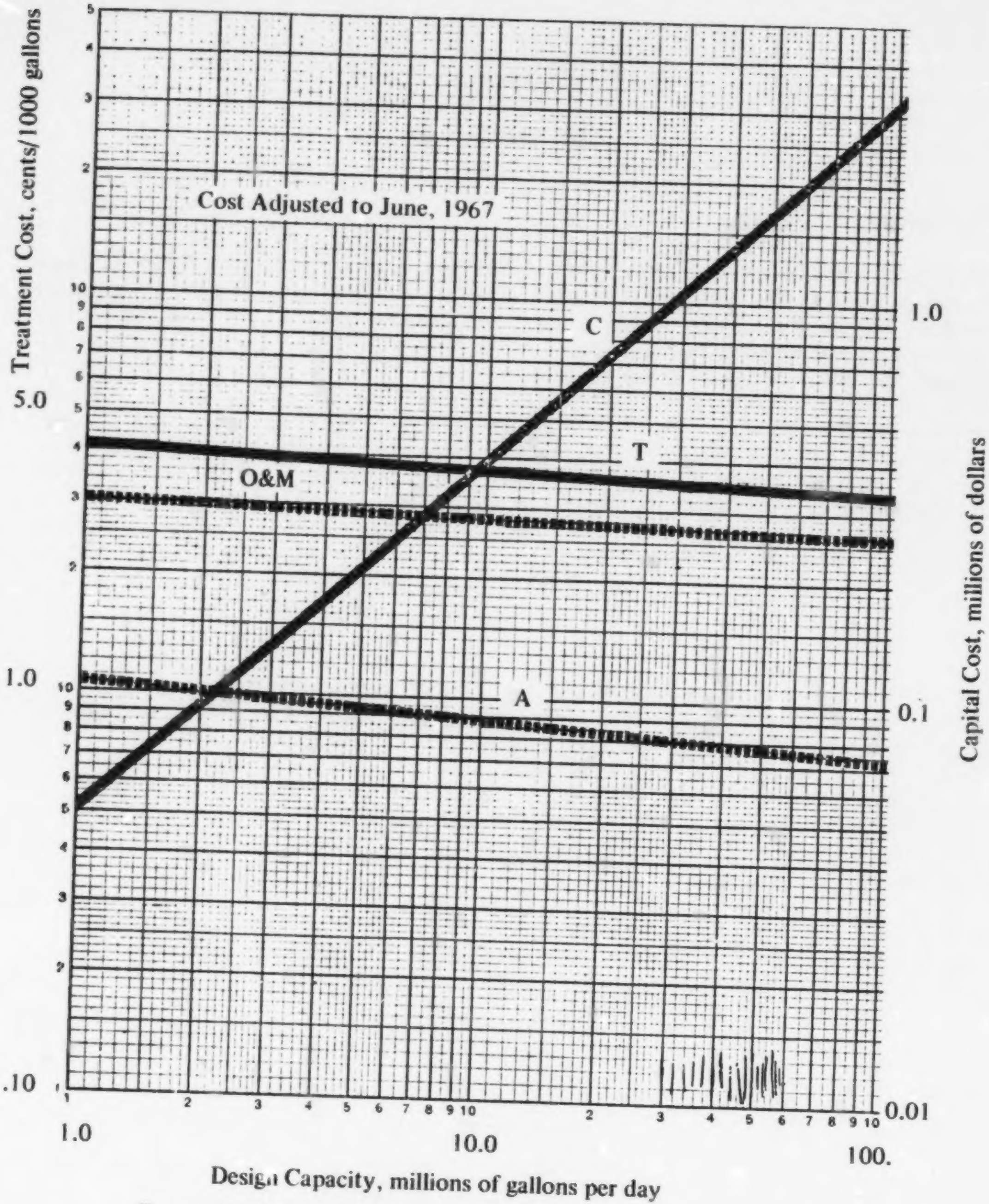
where:

C = capital cost in dollars

SA = surface area in $\frac{\text{square feet}}{1000}$

Their estimate for secondary clarifiers (overflow rate = 750 gpd/sf) was:

$$C = SA \left\{ 16.2 + \frac{6.9}{(SA)^{1.13}} \right\}$$



C = Capital Cost, millions of dollars
A = Debt Service, cents per 1000 gallons (4-1/2% - 25 yr.)
O&M = Operating and Maintenance Cost, cents per 1000 gallons
T = Total Treatment Cost, cents per 1000 gallons

Figure 11-1. SOLIDS REMOVAL BY COAGULATION & SEDIMENTATION

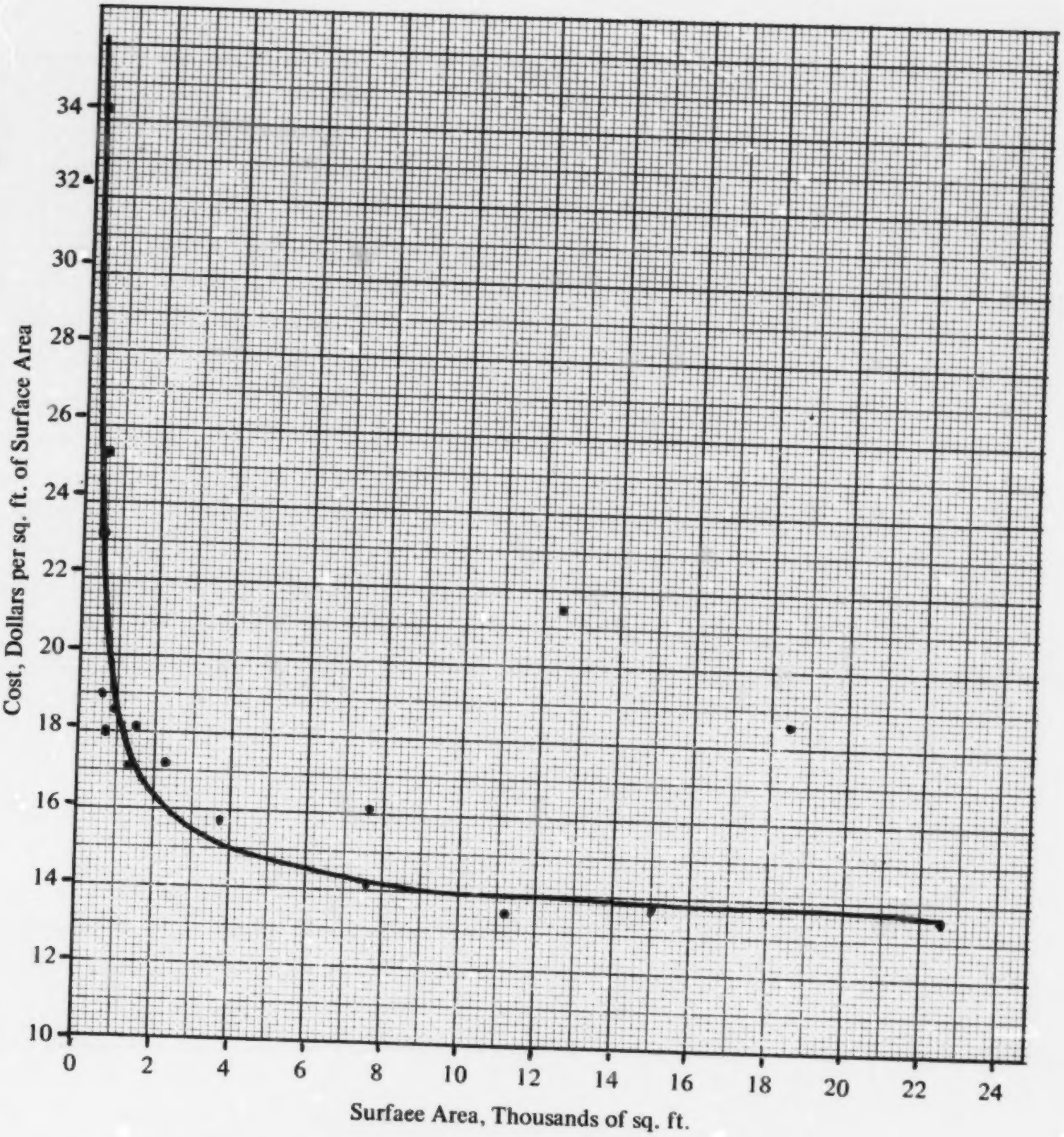


Figure 11-2. COST OF PARIMARY SEDIMENTATION TANKS
January 1960 ENR Index = 812

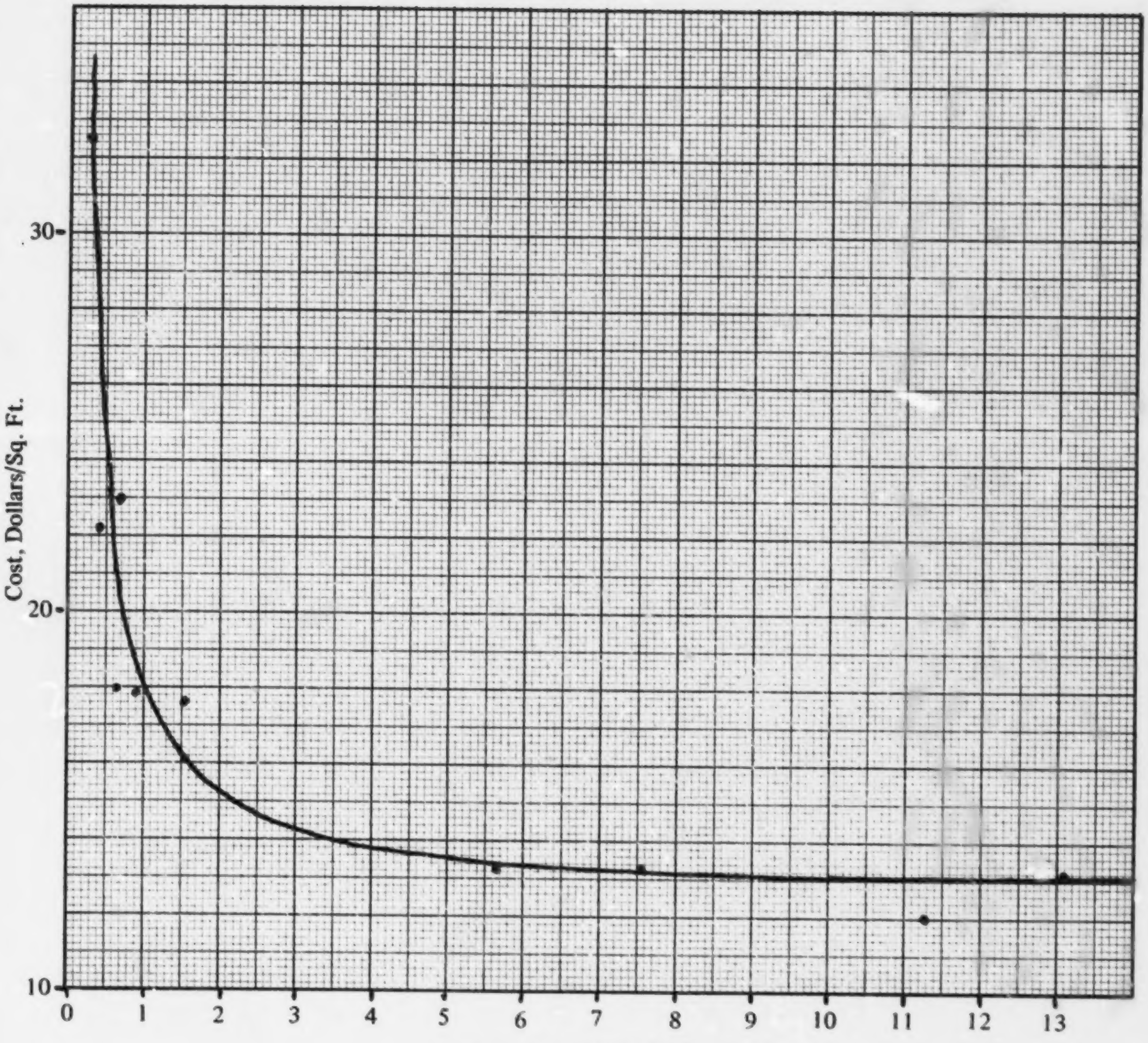


Figure 11-3. COST OF FINAL SETTLING TANKS
January 1960 ENR Index = 812

Tube settlers have been estimated to cost from $12 to $20/sq ft; with installation costs from $5 to $15/sq ft by one manufacturer (6). Some other manufacturers estimates are shown in Figure 11-4, representing capital cost of purchase and installation in existing basins (7), (8).

Smith and McMichael (2) estimated the costs of microscreening of activated sludge secondary effluent, as shown in Figure 11-5. Some manufacturers' estimates of capital costs of microscreening, including installation, but not housing, are depicted in Figure 11-6 (9),(10), (11), (12). Lynam, et al (13), estimated the total cost of microstraining at 10 MGD capacity based on their Chicago study to be 2.9 cents/1000 gallons.

One estimate of the cost of filtration through sand at 4 gpm/sq ft is illustrated in Figure 11-7 (2). A more recent estimate for mixed-media filters at the same hydraulic loading varies from 8.0 to 1.6 cents/1000 gallons at 1 and 100 MGD, respectively (4). Lake Tahoe costs have been computed at 4.1 cents/1000 gallons for mixed-media filters operated at approximately 5 gpm/sq ft. Some estimates of installed costs by manufacturers, excluding the enclosure facility, are illustrated in Figure 11-8 (6), (7), (8).

The moving-bed filter capital cost has been estimated by the manufacturer as shown in Figure 11-9.

11.3 Data Analysis

In order to normalize data presented, some of the necessary indices are presented in Tables 11-1 through 11-3. Table 11-1 shows a series of indices available for data analysis. Tables 11-2 and 11-3 provide monthly tabulations of the two indices which are most applicable for normalization calculations. Regional indices must also be applied. Standard amortization tables are available in a number of textbooks and handbooks.

11.4 Cost Estimation and Design

Some of the cost factors involved in the design or redesign of a treatment facility are: cost of design studies, type and strength of waste to be treated, current and projected population to be served, sewers, lift stations and transmission lines required, types of processes to be used, current and projected costs of land, personnel funds and chemicals, maintenance and service contracts and/or personnel costs, degree of treatment required and anticipated and cost of equipment and construction. In addition to these more or less concrete values, there are less predictable, but equally real, factors of political mood, funds, operating personnel criteria and availability and increasingly stringent effluent criteria.

A complete cost estimate should generally begin with the statement of purpose and scope; that is, "what is to be accomplished." Estimated costs of the projected installation should be broken down into process costs and ancillary facilities, with each unit process cost evaluation developed separately. All cost estimates should be related to one area and time to facilitate updating of those costs according to suitable indices. All equipment should be grouped according to function. Careful consideration should be given to installation cost factors for each piece of equipment, such as jobsite handling, special installation equipment or structures, assembly costs, etc. Associated work items, such as foundations, monitoring and control instrumentation and chemicals storage must be considered and included. Process piping and electrical work, i.e., all necessary appurtenances required to interconnect the various unit processes, should be considered separately.

The cost of enclosing and support buildings and structures cannot be ignored in the overall and total cost estimate. Cost factors such as lighting, low voltage power supplies, fire and burglar alarms and additional line power supplies must also be included.

The capital cost estimate should be followed by an equally detailed annual operating estimate if a true overall operating cost picture is to be generated.

Funds should be made available to have professional engineers handle all preliminary, design and construction phases.

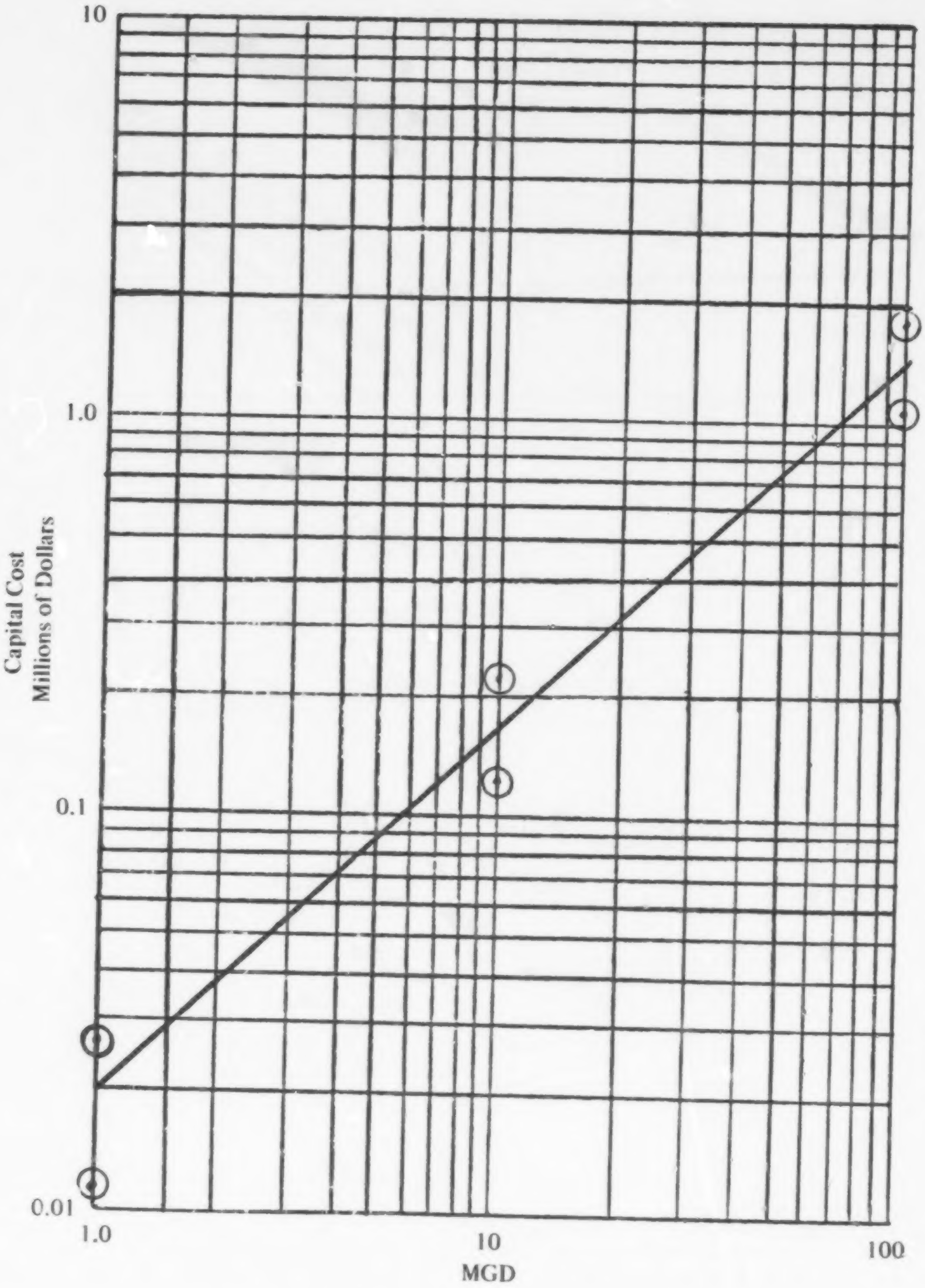
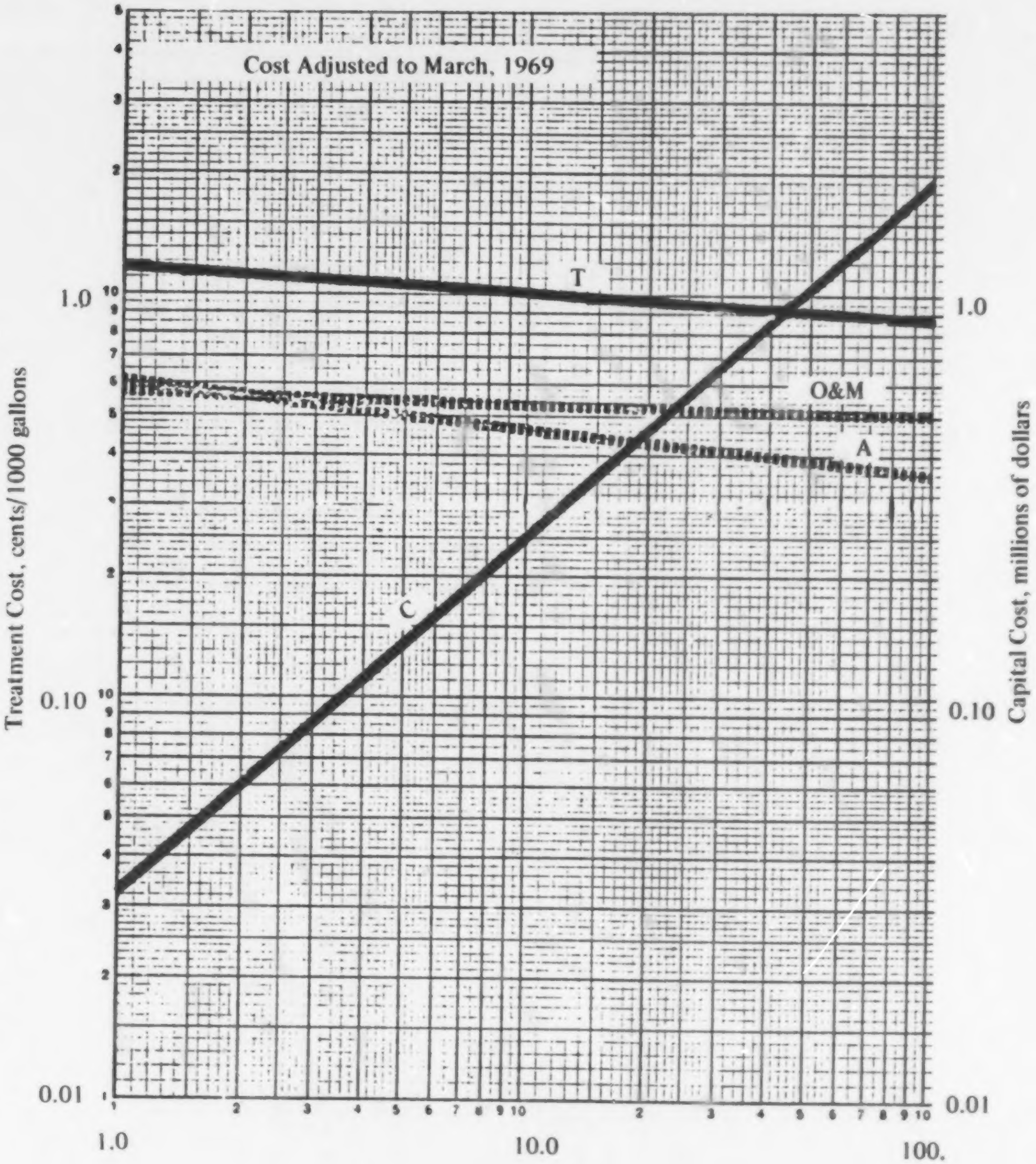


Figure 11-4. CAPITAL COST - TUBE SETTLER



Design Capacity, millions of gallons per day

C = Capital Cost, millions of dollars
A = Debt Service, cents per 1000 gallons (4-1/2% - 25 yr.)
O&M= Operating and Maintenance Cost, cents per 1000 gallons
T = Total Treatment Cost, cents per 1000 gallons

Figure 11-5. MICROSCREENING OF SECONDARY EFFLUENT

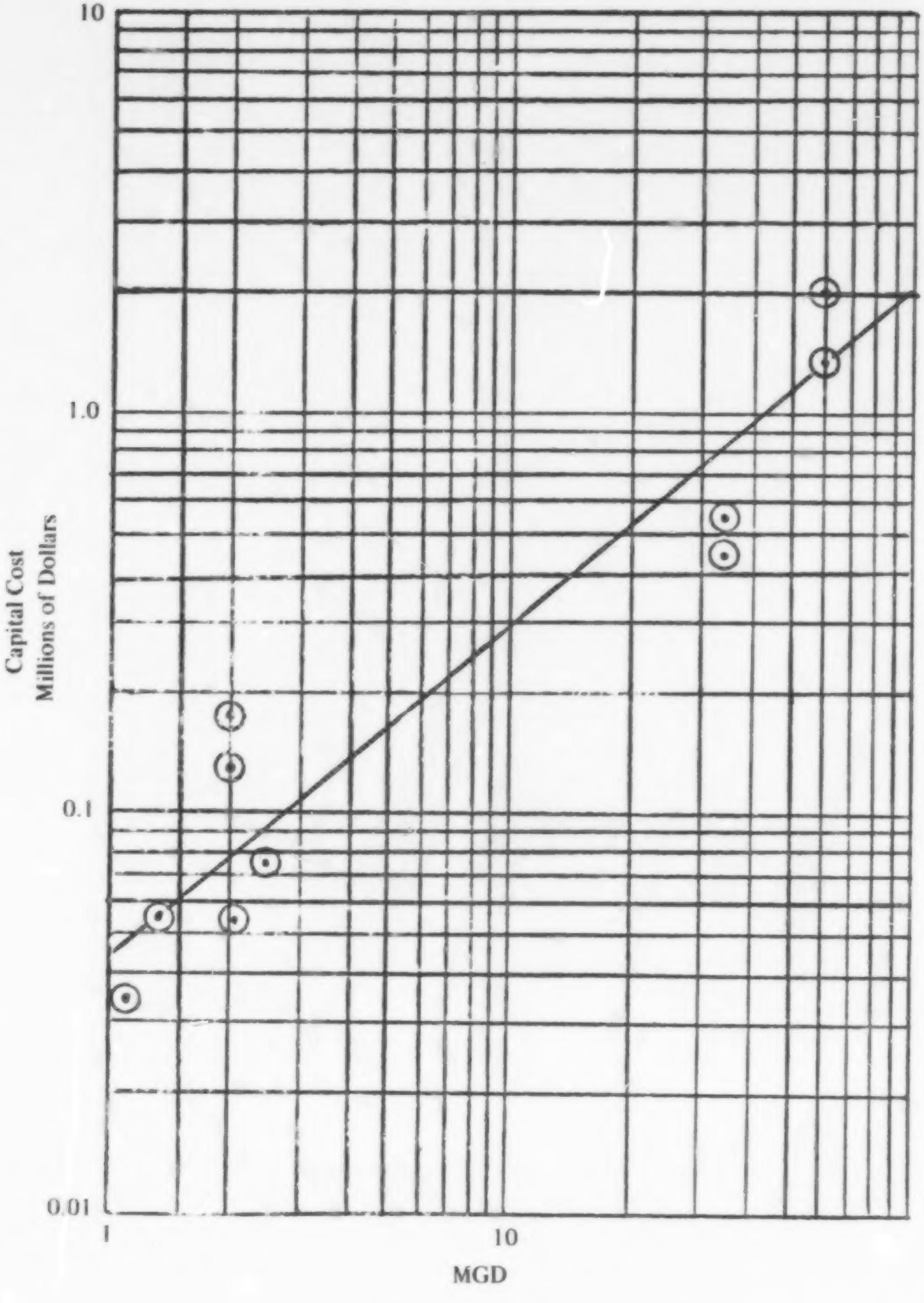
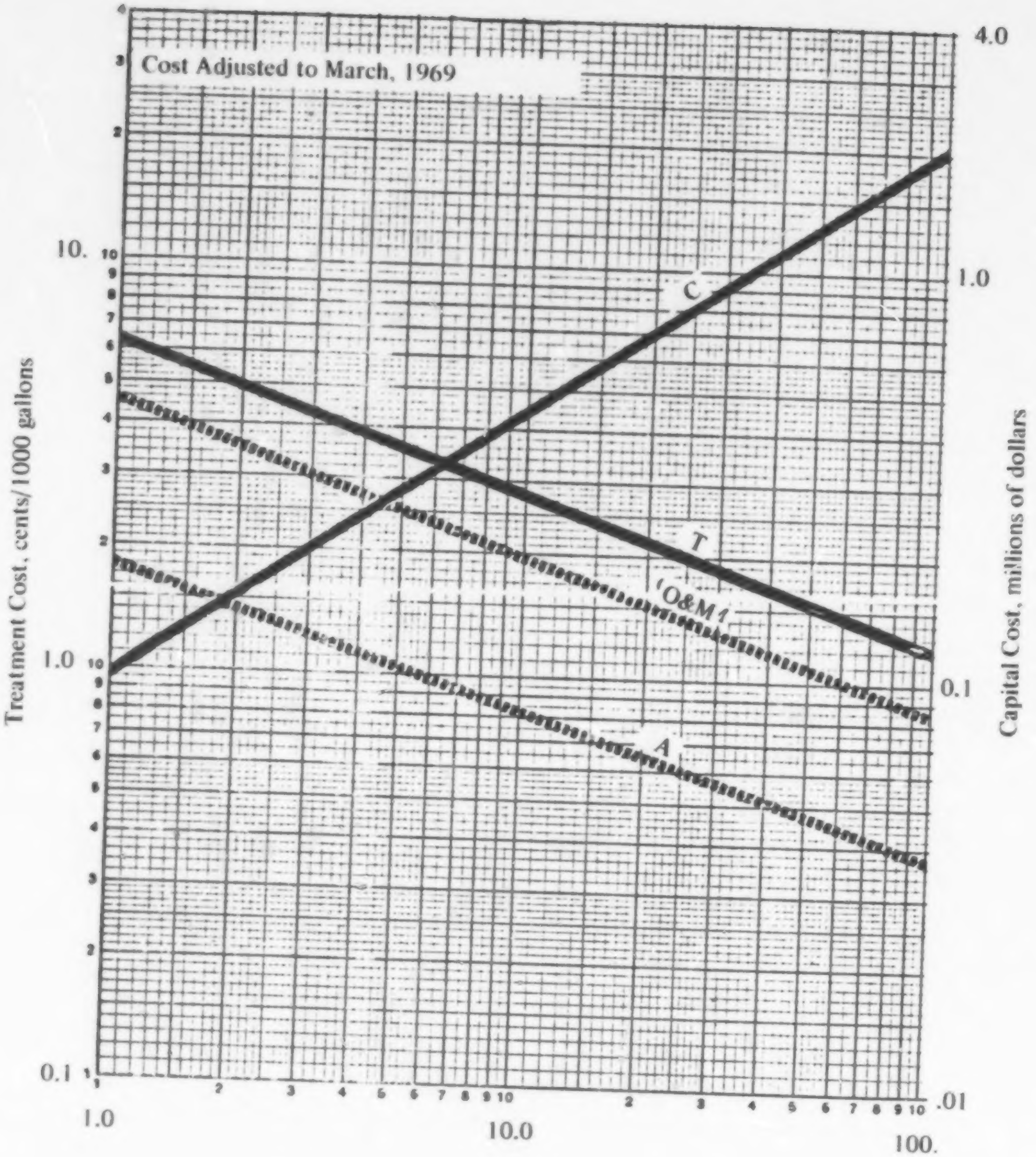


Figure 11-6. CAPITAL COSTS - MICROSCREENING



C = Capital Cost, millions of dollars
A = Debt Service, cents per 1000 gallons (4-1/2% - 25 yr.)
O&M= Operating and Maintenance Cost, cents per 1000 gallons
T = Total Treatment Cost, cents per 1000 gallons

Figure 11-7. FILTRATION THROUGH SAND OR GRADED MEDIA - 4GPM/SQ FT

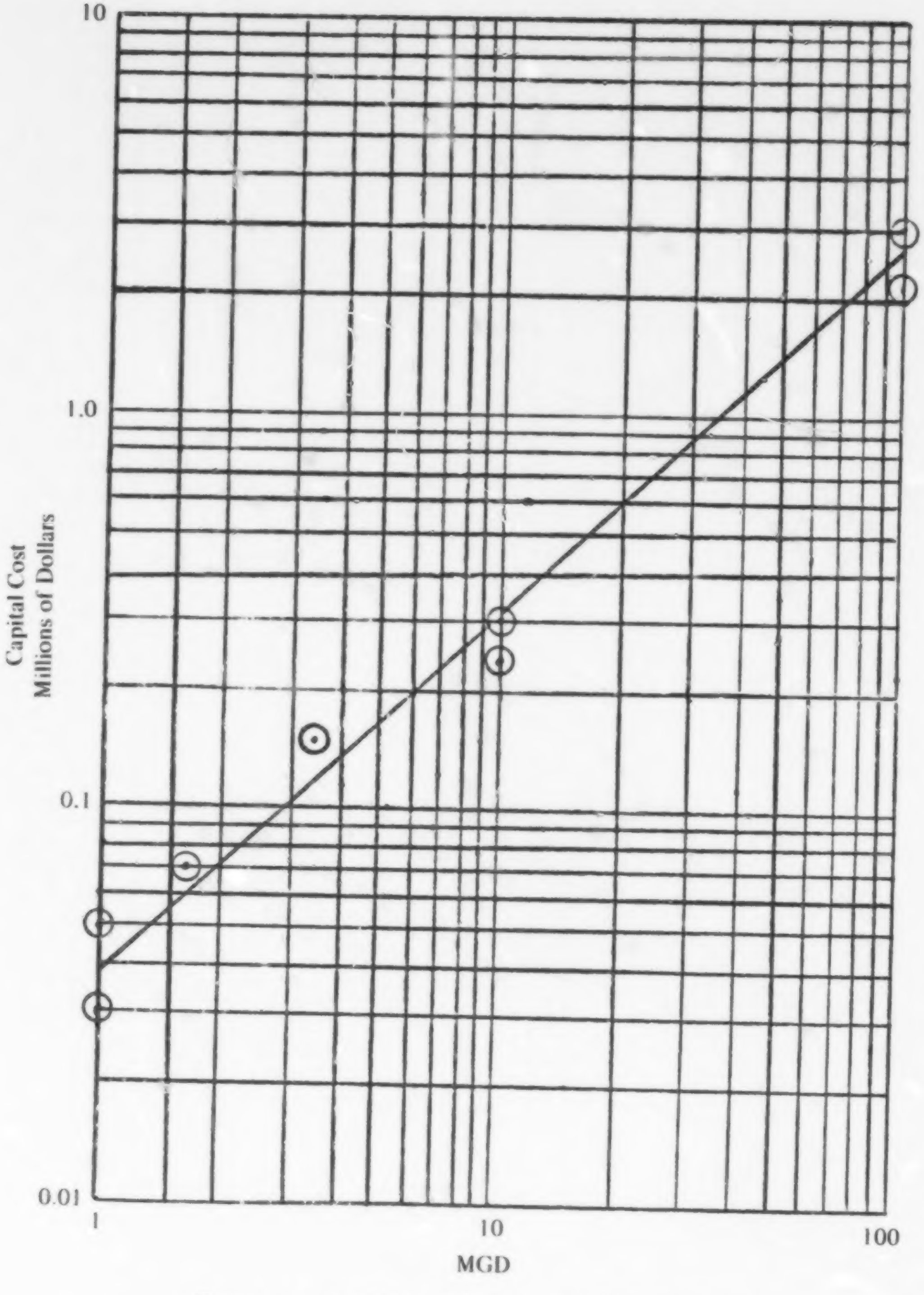


Figure 11-8. CAPITAL COST - MULTIMEDIA FILTER

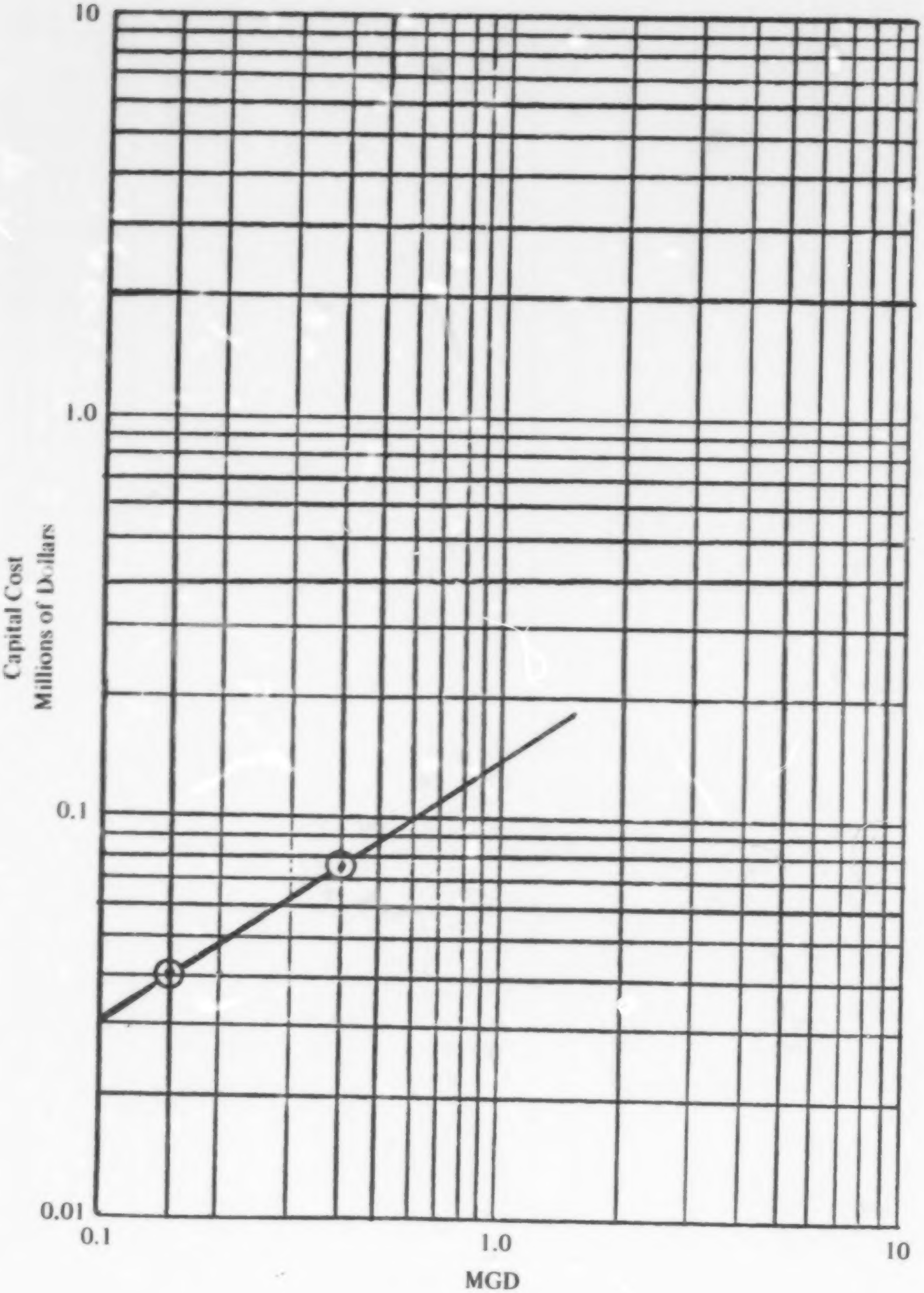


Figure 11-9. CAPITAL COST - MOVING BED FILTER

Table 11-1
YEARLY COST INDICES

	Marshall & Stevens installed-equipment indices		Engineering News-Record Construction index	Handy-Whitman index for water treatment plants[1]		Engineering News-Record Building cost index	Chemical Engineering plant instruction index
	1926 = 100		1913 = 100	1936 = 100		1913 = 100	1957 - 1959 = 100
Year	All Industry	Process Industry		Large Plant	Small Plant		
1956	209	206	690	275	276	491	94
1957	225	224	724	288	289	509	99
1958	229	228	759	296	296	525	100
1959	235	232	797	311	309	548	102
1960	238	237	824	317	317	559	102
1961	237	236	847	315	315	568	101
1962	239	237	872	324	322	580	102
1963	239	238	901	330	327	594	102
1964	242	241	936	340	336	612	103
1965	245	244	971	350	346	627	104
1966	252	252	1021	368	362	652	107
1967	263	256	1043[2]	380[2]	274[2]	660	110
1968	273	270	1111[2]			695	114
1969	285	282	1206[2]			760	119
1970	294[3]	292[3]	1311[2]			801	123[2]

1. Based on July of the year.
2. Based on January of the year.
3. Based on the first quarter of the year.

YEAR	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	ANNUAL INDEX
1957													98.04
1958													101.50
1959													103.65
1960													104.96
1961													105.83
1962									107.19	107.20	107.03	106.84	106.99
1963	106.80	107.05	107.08	107.11	107.22	107.78	108.07	108.52	108.58	109.54	109.51	109.60	108.52
1964	109.64	109.45	109.53	109.57	109.70	109.99	110.24	110.54	110.63	110.69	110.73	110.68	110.11
1965	110.82	111.04	111.07	111.12	111.15	111.83	112.31	112.57	112.70	112.82	112.87	113.09	111.95
1966	114.05	114.60	114.77	115.08	115.34	116.05	116.82	116.92	117.11	117.51	117.46	117.48	116.10
1967	117.76	118.08	118.11	118.22	118.34	119.11	119.63	120.28	120.59	120.89	120.91	121.01	119.41
1968	121.10	121.20	121.21	121.55	121.71	122.49	123.39	123.69	124.53	126.80	127.24	127.71	123.55
1969	128.68	129.50	129.84	130.03	130.03	131.11	132.44	135.34	135.46	135.85	136.61	136.86	132.65
1970	137.63	137.87	138.15	138.49	141.18	143.03	146.25	146.70	147.45	148.07	149.28	149.63	143.64
1971	150.60	150.89	153.34	155.41	157.29								

Table 11-2 SEWAGE TREATMENT PLANT CONSTRUCTION COST INDEX

Source: Sewer and Sewage Treatment Plant Construction Cost Index, FWPCA, Dept. of Interior, Dec. 1967

Table 11-3

LABOR COST INDEX

WATER, STEAM, AND SANITARY SYSTEMS NONSUPERVISORY WORKS, $/hr.

YEAR	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	ANNUAL AVERAGE
1958	1.96	2.00	1.97	1.96	1.96	2.00	2.00	2.00	2.02	2.03	2.08	2.05	2.00
1959	2.07	2.06	2.05	2.02	2.03	2.04	2.06	2.03	2.09	2.10	2.13	2.09	2.07
1960	2.11	2.12	2.12	2.14	2.13	2.15	2.18	2.18	2.18	2.21	2.23	2.22	2.17
1961	2.25	2.27	2.26	2.27	2.26	2.26	2.29	2.27	2.29	2.29	2.30	2.29	2.27
1962	2.28	2.30	2.30	2.32	2.31	2.33	2.34	2.32	2.34	2.33	2.37	2.36	2.33
1963	2.37	2.38	2.37	2.37	2.35	2.37	2.38	2.38	2.39	2.40	2.42	2.43	2.38
1964	2.42	2.44	2.43	2.44	2.44	2.44	2.44	2.43	2.46	2.45	2.48	2.47	2.44
1965	2.49	2.51	2.50	2.51	2.51	2.53	2.54	2.55	2.56	2.55	2.59	2.58	2.54
1966	2.62	2.65	2.63	2.67	2.66	2.65	2.69	2.67	2.70	2.72	2.74	2.74	2.68
1967	2.76	2.78	2.77	2.79	2.80	2.81	2.83	2.81	2.86	2.86	2.88	2.85	2.82
1968	2.89	2.91	2.91	2.95	2.97	2.98	3.03	3.01	3.05	3.08	3.11	3.00	2.99
1969	3.14	3.13	3.14	3.17	3.17	3.20	3.23	3.26	3.29	3.30	3.38	3.47	3.24
1970	3.47	3.44	3.44	3.46	3.48	3.48	3.52	3.53	3.56	3.58	3.62	3.59	3.51
1971	3.68	3.68	3.64										

Source: U. S. Department of Labor, Bureau of Labor Statistics, Employment and Earnings Statistics for the United States, 1909-67 (Bulletin No. 1312-5).

11.5 References

1. Smith, R., "Cost of Conventional and Advanced Treatment of Wastewater", Jour. WPCF, 40, 1546 (Sept. 1968).

2. Smith, R., and McMichael, W.F., "Cost and Performance Estimates for Tertiary Wastewater Treating Processes", USDI, FWPCA Report No. TWRC-9, (June 1969).

3. Evans, D.R., and Wilson, J.C., "Actual Capital and Operating Costs for Advanced Waste Treatment", Paper presented at WPCF 43rd Annual Conference, Boston, Mass. (Oct. 1970).

4. Roesler, J.F., Private Communication.

5. Barnard, J.L., and Eckenfelder, W.W., Jr., "Treatment Cost Relationships for Organic Industrial Wastes", Paper presented at 5th International Water Pollution Research Conference, San Francisco, California (July 1970).

6. Neptune - Microfloc, Inc., Corvallis, Oregon.

7. Permutit Co., Paramus, N.J.

8. Graver Water Conditioning Company, Union, N.J.

9. Crane Company - Cochrane Division, King of Prussia, Pa.

10. Marske, D.M., "High-Rate, Fine Mesh Screening of Combined Wastewater Overflows", Jour. WPCF, 42, 1476 (Aug. 1970).

11. Walker Process Equipment, Aurora, Illinois.

12. Zurn Industries, Inc., Water and Waste Treatment Div., Erie, Pa.

13. Lynam, B., Ettelt, G., and McAloon, T., "Tertiary Treatment at Metro Chicago by Means of Rapid Sand Filtration and Microstrainers", Jour. WPCF, 41, 247 (Feb. 1969).

14. Johns-Manville, New York, N.Y.

APPENDIX A

GLOSSARY

Adsorption – The taking up of a gas, vapor or dissolved material on the surface of a solid.

Aerate – To impregnate or saturate a liquid with air.

Aerobic – In the presence of oxygen.

Aggregate – Particles brought together; agglomeration.

Angle of Repose – That angle at which a material will just rest upon a surface without internal shearing.

Alum – Aluminum Sulfate – $Al_2(SO_4)_3 \cdot X\,H_2O$

Alumina – Aluminum oxide (Al_2O_3)

Anaerobic – In the absence of oxygen.

Anhydrous – Dry or devoid of water.

Baumé Scale – A measure of the density of a solution heavier than water; Degrees Baumé

$$(^{\circ}Be) = 145 - \frac{145}{\text{Specific Gravity}}$$

Biochemical Oxygen Demand – The amount of oxygen required for the biological oxidation of the organic matter in a liquid over a stated period of time.

Brownian Movement – The random motion of colloidally dispersed particles as a result of collisions with the molecules of the dispersing medium.

Chemical Oxygen Demand – The amount of oxygen required for the chemical oxidation of organics in a liquid.

Coagulation – Process by which chemicals (coagulants) are added to an aqueous system for the purpose of creating rapid-settling aggregates out of finely divided, dispersed matter with slow or negligible settling velocities. Forces causing particles to repel each other are neutralized by the coagulants.

Colloid – A dispersion of very small (1 mμ - 0.5 μ) particles suspended in a fluid.

Contact Clarification – A process consisting of the rapid mixing of chemically-treated waste with previously settled floc, the settled floc constituting a surface for subsequent floc formation.

APPENDIX A (Continued)

GLOSSARY

Detention Time – The average period of time that a fluid element is held in a basin or tank before discharge.

Dolomitic Lime – Calcium oxide plus variable quantities of magnesium oxide, typically 34-40% MgO.

Dual Media Filtration – A filtration process that uses a bed composed of two distinctly different substances, such as anthracite coal and sand, as opposed to conventional filtration through sand only.

Floc – An agglomeration of finely-divided or colloidal particles due to certain chemical-physical or biological operations.

Flocculation – A gentle agitation of waste water for a period of time so as to increase the number of collisions between the smaller floc particles, causing them to stick together and grow into large readily-settleable masses.

Heat of Solution – The amount of heat liberated or absorbed from dissolving a chemical in water or other solvent.

Hydrated Lime – Calcium Hydroxide: $Ca(OH)_2$

Hydrophilic – "Water loving"; having a high affinity for water.

Hydrophobic – "Water hating"; having a low affinity for water.

Hygroscopic – A solid with the ability to absorb moisture from the air without eventual dissolution in that moisture.

Hyperfiltration – A high-pressure reverse osmosis process using a membrane which will remove dissolved salts as well as suspended solids.

Ions – An atom or group of atoms with an unbalanced electrostatic charge.

Microscreening – A form of surface filtration using specially woven wire fabrics mounted on the periphery of a revolving drum.

Millimicron – 10^{-7} centimeters.

Mixed Liquor Suspended Solids – (MLSS) - The concentration of suspended solids carried in the aeration basin of an activated sludge process.

pH – Measure of acidity or basicity, expressed as the negative logarithm of the hydrogen ion concentration.

APPENDIX A (Continued)

GLOSSARY

Polyelectrolytes – Chemicals that consist of high molecular weight molecules with many reactive groups situated along the length of the chain. Polyelectrolytes react with the fine particles in the waste and assist in bringing them together into larger and heavier masses for settling.

Precoat -- The initial coating of the septum in a diatomaceous earth filter to provide the initial straining medium.

Quicklime – Calcium oxide (CaO) or calcium oxide in natural association with a lesser amount of magnesium oxide.

Recalcination – A process or recovering lime for reuse by means of heating spent lime sludge to high temperatures, thereby driving off water of hydration and carbon dioxide.

Recarbonation – The addition of carbon dioxide to lime-treated water to effect a downward adjustment of the pH of the waste for further calcium removal and/or stabilization of the water.

Soda Ash – Sodium carbonate – Na_2CO_3

Sludge Volume Index – The volume in milliliters of one gram of activated sludge in the mixed liquor which has settled for thirty minutes; an indication of the settling characteristics of the secondary sludge.

Specific Gravity – The ratio between the weight of a substance and the weight of an equal volume of water at any stated temperature.

Turbidity – Optical measure of suspended matter in water.

Unit Process – A process which can be clearly distinguished from other processes in a treatment system.

Weir Loading – An index of settling basin effluent hydraulics that is determined by dividing the total daily flow by the total weir length.

ACKNOWLEDGMENT

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A-4

CHAPTER 12

PLAN AND SPECIFICATION REVIEW

CHECK SHEET

12.1 General

The processes outlined in this design manual have been developed for suspended solids removal from wastewaters. In some cases, these processes have had limited use in full-scale design of wastewater treatment facilities. Design engineers and reviewing authorities may not be completely familiar with the parameters to be considered in the preparation of plans and specifications for a given project. The following plan and specification review check sheet has been prepared to serve as a guide to the engineer in the design of proposed facilities which use the process or processes outlined in this manual. It will also be used by the Environmental Protection Agency, and may be used by State and local authorities in their review of projects for approval.

As with the case of any check sheet, its purpose is to fully consider all possible parameters for an individual process. For a given project all or part of the check sheet may be applicable and should be used with this fact in mind.

12.2 Plan Review Data Sheet for Suspended Solids Removal

Applicant ____________________ Project No. ____________________

Design Engineer ____________________ Address ____________________

Phone ____________________

Review Engineer ____________________ 1st Review Date ____________________

Return for Revision ____________________ Revision Receipt Date ____________________

Revision Approval Date ____________________

Date of State Approval ____________________

Type of Work:

Addition to Existing Facility

New Facility

Primary ☐

Secondary ☐

Tertiary ☐

In the space provided below furnish a simplified flow diagram or a written description of the plant units in the flow sequence. Include the method of sludge handling and ultimate disposal.

Sedimentation

No. tanks ______________________ Shape of tanks ______________________

Dimensions: Length ______ Width ______ Diameter ________ Depth __________

Vol. each tank _____ cu ft Total vol. ____________ cu ft

Ave. flow: Present _________ MGD; Design ________ MGD

Detention time: Present _______; Design _________________

Overflow rate: Present ________ MGD/ft^2 Design ________ gpd/ft^2

Length of weir _____________; Location ______________

Weir loading: Present _______ gpd/ft; Design ________ gpd/ft

Scum removal: Method ___________ Disposition ___________

Sludge removal: Method ___________ Disposition ___________

Sketch – Plan and/or section of tank

High-Rate Settler

Type: Tubes ☐ Plates ☐

Unit in which installed ____________________

Area of settling tank ____________________ sq ft

Surface area of high-rate settler ____________________ sq ft

Length of tube or plate: ____________________

Sketch opening shape and dimensions: ____________________

Angle of inclination from horizontal: ____________________

Direction of feed versus sludge flow: ☐ Cocurrent

☐ Countercurrent

Material of tube or plate:

Rate of flow __________ gpm/sq ft (end area)

Volume throughput of high-rate installation ______ MGD

Chemical Clarification

a) Chemical Addition

Chemicals used: a. ______; b. ______; c. ______

Chemical storage area: __________ sq ft

Chemical storage volume: __________ cu ft

		a	b	c
Chemical day tanks:	Material	______	______	______
	Volume	______	______	______

Feed equipment: Manufacturer, size and capacity

a __________ gpd/ppd
b __________ gpd/ppd
c __________ gpd/ppd

Point of application:

a __________
b __________
c __________

b) Mixing of Chemicals and Wastewater

Mechanical: describe size and capacity ____________________________
____

Hydraulic: ____

Location of mixing area: Pipe ______________________________
Tanks ______________________________
Supplemental tank ______________________

Tank size: Length ____ Width ____ Diameter ____ Depth ____

c) Flocculation

Type of chamber: Baffle
Mechanical

Dimensions of tank: Diameter ____ Length ____ Width ____ Depth ______

Detention time for floc growth ________________

Number of baffles: V/H ________________

Number of mixers:

Type of mixer: HP ________________

Solids-Contact Clarifier

Number of tanks __________ ; Shape of tanks ____________________

Tank construction ______________________________________

Dimension: Length _____ Width ______ Diameter ______ Depth __________

Volume each tank: ______ cu ft Total volume: ______ cu ft

Average flow/tank: Present ________ MGD; Design ________ MGD

Average detention time: ________ ; Max. ______ ; Min. ______

Overflow rate: ______________ gpd/sq. ft.

Chemical feed application ______________________________

Mixing and flocculating: Volume ____________ cu ft

Mixer __________ HP
Detention time ______________

Clarification area/volume: __________ __________ cu ft

Detention time __________

Solids recirculation ______________________

Sludge collector ____________; Sludge thickener ______________

Head available: ____________ ft

Maximum headloss ______________________________

Flow inlet: ______ in dia Flow outlet: ______ in dia

Collector trough (weir loadings): __________ gpd/ft

Flow control ______________________________

Microscreens

Number of units ______ ; Self-contained _____ ; Concrete structure __________

Capacity of unit: ______ MGD; Total capacity: ____ MGD

Screen size ________ ; Number of screen elements ______________

Overall size of screen: Length ______ ; Diameter ______________

Installation dimensions of single unit: Length ___ Width ___ Height _____

Flow rate _____ gpm/sq ft of submerged area

Drive motor: ________ HP ________ Variable speed ________

Wash pump: ________ gpm; HP ________________

Waste water return pump (variable speed): _____ gpm HP _______

Screening:__________; Opening size in microns: ____________________

Valved overflow line: Size__________ Material ____________________

Material of construction: __________ Screen _____; Bearings __________

Pipe __________; Backup __________; Waste collector __________

Screen housing _____; Shaft __________; Splash guards __________

Inlet weir __________; Outlet ____________________

Nozzles: No. _____; Size ____________________

Instrumentation ____________________

Head differential indicator ____________________

Level control indicator ____________________

Automatic flow control: Speed
Backwash

Recorder ____________________

High level alarm ____________________

Corrosion control and coating ____________________

Moving Bed Filters

Number of units:______2 bed configuration
______4 bed configuration

Total number of units __________

Total flow filtering area: __________ sq ft ____________________

Flow/unit: __________ gpm _____; Total flow: __________ gpm __________

Sand filtering media: __________ e.s. __________; u.c. __________

Filter bed size: __________ Dia. __________; Length __________

Tank material ____________________

Discharge screen material ____________________

Overall module dimension: Length ________ Width ________ Height ___________

Head tank: ___________ Dia. ________; Depth ________

Residence time: ______ minutes ______________

Headloss __________ ; Feed control __________________

Level control ____

Sludge settling tank: Diameter _______ ; Depth __________

Detention time: ____ minutes

Volume: __________ cu ft

Collection equipment _______ ; Disposition _____________

Chemical system: _____ Coagulant; _______ Polyelectrolyte

Storage tank dia depth, material ____

Day tank dia depth, material ____

Feeders: Rate and control ____

Mixers ____

Sand washer tank: Dia. ________ : Depth _______ : Volume ______________

Air: ___________ cfm; Water: _______ gpm
___________ psi _______ psi

Materials of construction

Tanks ____

Hopper bottoms ____

Diaphragm ____

Sand educators ____

Pipe ____

Pumps and motors: Type: ______ Material: ________ HP and capacity _______

Sand movement ____

Cutter ____

Sand cleaning ____

Screen wash ____

Chem. pump and mixer ____

Air compressor ____

Operational controls ____

Filters

Type: Pressure ____________ ; Gravity ____________

Dual Media _________ ; Multi Media _______ ; Graded Sand ___________

Number of filters ____

Dimensions: Length _______ ; Width/Diameter _______ ; Depth ________

Volume each filter: ______ cu ft Total volume: _______ cu ft

Average flow/unit: __________ gpm

Total flow: __________ gpm; ____________ gpd

Strainers _____________ ; Retaining Plate ______________

Sand ___________ in. depth: Size ________ e.s. _________ u.c.

Gravel _________ in. depth; Size ________

Anthracite ________ in. depth; Size ________ e.s. _________ u.c.

Garnet __________ in. depth; Size ________ e.s. _________ u.c.

Control valves ____

Level control ____

Chemicals: ______________________ Coag. __________ ; Poly. _________________

Day tank: ________ Dia. _____ Depth _____ Dia. _____ Depth ________

Feeding equipment ____

Pt. of application ____

Backwash operation: ____

Surface wash ______ Air ________________

Backwash rate:

Collection of waste water ________________

Point of waste water disposal _____________